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# TRANSPORTATION RESEARCH RECORD

Journal of the Transportation Research Board, No. 2149

## Traffic Control Devices, Visibility, and Highway–Rail Grade Crossings 2010

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*OF THE NATIONAL ACADEMIES*

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TRANSPORTATION RESEARCH BOARD
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# Foreword

The 2010 series of the *Transportation Research Record: Journal of the Transportation Research Board* consists of approximately 900 papers selected from 3,700 submissions after rigorous peer review. The peer review for each paper published in this volume was coordinated by the committee acknowledged at the end of the text; members of the reviewing committees for the papers in this volume are listed on page ii.

Additional information about the *Transportation Research Record: Journal of the Transportation Research Board* series and the peer review process appears on the inside back cover. TRB appreciates the interest shown by authors in offering their papers, and the Board looks forward to future submissions.

**Note:** Many of the photographs, figures, and tables in this volume have been converted from color to grayscale for printing. The electronic files of the papers, posted on the web at **www.TRB.org/TRROnline**, retain the color versions of photographs, figures, and tables as originally submitted for publication.

## Measurement Conversion Factors

To convert from the unit in the first column to the unit in the second column, multiply by the factor in the third column.

| *Customary Unit* | *SI Unit* | *Factor* |
|---|---|---|
| **Length** | | |
| inches | millimeters | 25.4 |
| inches | centimeters | 2.54 |
| feet | meters | 0.305 |
| yards | meters | 0.914 |
| miles | kilometers | 1.61 |
| **Area** | | |
| square inches | square millimeters | 645.1 |
| square feet | square meters | 0.093 |
| square yards | square meters | 0.836 |
| acres | hectares | 0.405 |
| square miles | square kilometers | 2.59 |
| **Volume** | | |
| gallons | liters | 3.785 |
| cubic feet | cubic meters | 0.028 |
| cubic yards | cubic meters | 0.765 |
| **Mass** | | |
| ounces | grams | 28.35 |
| pounds | kilograms | 0.454 |
| short tons | megagrams | 0.907 |
| **Illumination** | | |
| footcandles | lux | 10.76 |
| footlamberts | candelas per square meter | 3.426 |
| **Force and Pressure or Stress** | | |
| poundforce | newtons | 4.45 |
| poundforce per square inch | kilopascals | 6.89 |
| **Temperature** | | |
| To convert Fahrenheit temperature (°F) to Celsius temperature (°C), use the following formula: °C = (°F – 32)/1.8 | | |

| *SI Unit* | *Customary Unit* | *Factor* |
|---|---|---|
| **Length** | | |
| millimeters | inches | 0.039 |
| centimeters | inches | 0.394 |
| meters | feet | 3.281 |
| meters | yards | 1.094 |
| kilometers | miles | 0.621 |
| **Area** | | |
| square millimeters | square inches | 0.00155 |
| square meters | square feet | 10.764 |
| square meters | square yards | 1.196 |
| hectares | acres | 2.471 |
| square kilometers | square miles | 0.386 |
| **Volume** | | |
| liters | gallons | 0.264 |
| cubic meters | cubic feet | 35.314 |
| cubic meters | cubic yards | 1.308 |
| **Mass** | | |
| grams | ounces | 0.035 |
| kilograms | pounds | 2.205 |
| megagrams | short tons | 1.102 |
| **Illumination** | | |
| lux | footcandles | 0.093 |
| candelas per square meter | footlamberts | 0.292 |
| **Force and Pressure or Stress** | | |
| newtons | poundforce | 0.225 |
| kilopascals | poundforce per square inch | 0.145 |
| **Temperature** | | |
| To convert Celsius temperature (°C) to Fahrenheit temperature (°F), use the following formula: °F = (°C × 1.8) + 32 | | |

## Abbreviations Used Without Definitions

| | |
|---|---|
| AASHO | American Association of State Highway Officials |
| AASHTO | American Association of State Highway and Transportation Officials |
| ACRP | Airport Cooperative Research Program |
| APTA | American Public Transportation Association |
| ASCE | American Society of Civil Engineers |
| ASTM | American Society for Testing and Materials (known by abbreviation only) |
| FAA | Federal Aviation Administration |
| FHWA | Federal Highway Administration |
| FMCSA | Federal Motor Carrier Safety Administration |
| FRA | Federal Railroad Administration |
| FTA | Federal Transit Administration |
| IEEE | Institute of Electrical and Electronics Engineers |
| ISO | International Organization for Standardization |
| ITE | Institute of Transportation Engineers |
| NASA | National Aeronautics and Space Administration |
| NCHRP | National Cooperative Highway Research Program |
| NHTSA | National Highway Traffic Safety Administration |
| RITA | Research and Innovative Technology Administration |
| SAE | Society of Automotive Engineers |
| SHRP | Strategic Highway Research Program |
| TCRP | Transit Cooperative Research Program |
| TRB | Transportation Research Board |

# Suggestions Concerning School Traffic Control Devices

Kay Fitzpatrick, Marcus A. Brewer, and Eun Sug Park

**School speed zones are frequently requested as traffic controls for school areas on the basis of the common belief that if a transportation agency would install a reduced speed limit, then drivers would no longer speed through an area. A Texas research project was initiated to review existing practices and to develop guidelines concerning the establishment of school zones. Researchers sought to answer several questions about the relationships between operating speed and reduced-speed school zones. Researchers documented existing knowledge through a review of previous research on the effectiveness of devices, surveys of practitioners and law enforcement officers, and a review of existing school zone guidelines and warrants. In addition, researchers collected speed data at 24 school zones, focusing the analysis on unique characteristics of reduced-speed school zones, such as length, duration, and time of day relative to the start and end of school. Of particular interest was the finding that longer school zones do not result in lower speeds for a longer distance; speeds increased approximately 0.9 mph for every 500 ft of school zone length. Findings from the analysis and discussions with practitioners were used to develop guidelines for traffic control devices, including school speed zones, near schools in Texas. The guidelines were designed to serve as a supplement to the *Texas Manual on Uniform Traffic Control Devices* and the manual *Procedures for Establishing Speed Zones.* This paper focuses on the findings from the field studies and presents suggestions concerning school traffic control devices.**

The state of Texas has experienced considerable population growth in recent years. This growth has produced new schools in areas near highways originally designed for low volume and relatively high speeds. Another trend is the higher proportion of children being transported to and from schools in automobiles. These realities, and many other issues associated with traffic around schools, make it important to periodically review the traffic control on roadways near schools to ensure a safe traffic environment.

Discussions with practitioners indicate that reduced-speed school zones are frequently requested traffic controls for school areas, on the basis of the common belief that if the transportation agency would install a reduced speed limit, then drivers would no longer speed through the area. Unfortunately, there are many situations in which a reduced-speed school zone is not the appropriate solution. The purpose of a recent Texas Department of Transportation (Texas DOT) Project, *Comprehensive Guide to Traffic Control near Schools,* was to examine traffic control treatments in use near schools, especially those associated with reduced-speed school zones located on higher-speed roadways (*1*). As a result of the investigation, the research team developed recommended guidelines for school zone traffic control devices. These guidelines are available on the Internet (*2*). The guidelines were designed to serve as a supplement to the *Texas Manual on Uniform Traffic Control Devices* (*3*) and the Texas manual *Procedures for Establishing Speed Zones* (*4*).

Researchers began the project by documenting existing knowledge on traffic control devices in school zones. This effort took several forms: a review of previous research studies examining effectiveness of devices, a survey of practitioners on signing and marking, a review of state and city school zone guidelines and warrants, and a telephone survey of law enforcement officers. By using this information, the research team identified several areas of emphasis for speed data collection. Researchers collected field data at school zones across Texas and analyzed the data for findings on speed–distance relationships, speed–time relationships, influences of site characteristics on speeds, and speed characteristics of school zones. The findings from these efforts are documented elsewhere (*1*). The objectives of this paper are to discuss the findings on vehicle speeds in school zones and to present recommendations for traffic control devices in school zones.

## STUDY METHODOLOGY

The research team wanted to collect speed data in school speed zones to determine the predominant speed patterns when the zones were active. Studies were conducted at 24 school sites. Elementary and middle schools located on higher-speed roadways were emphasized in the selection of study sites. Characteristics of the 24 sites are listed in Table 1.

### Data Collection

Researchers used laser guns and traffic counters to collect speed data. Lasers provided continuous speed measurements over a long distance in the zone for a short time period, and traffic counters provided several hours or days for selected spot locations in the zone. Laser data were collected in three periods: morning school zone (and immediately preceding, following, or both), noon-hour period, and afternoon school zone (and immediately preceding, following, or both). Each laser data collection period was approximately 1.5 to 2.5 h, depending on the duration of the active school zone. Researchers collected continuous speed for several hundred feet before, within, or following the school zone. Researchers selected only free-flowing passenger vehicles (e.g., semitrailers or buses were not selected).

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**TABLE 1 General Characteristics of Primary Roadway**

| Variable | Value | Freq. | % | Variable | Value | Freq. | % |
|---|---|---|---|---|---|---|---|
| Regulatory speed limit (RSL), mph | 30 | 1 | 4 | Number of lanes (num lanes)[a] | 2 + TWLTL | 2 | 8 |
| | 45 | 6 | 25 | | 2U | 6 | 25 |
| | 50 | 3 | 13 | | 3U | 1 | 4 |
| | 55 | 7 | 29 | | 4 + TWLTL | 8 | 33 |
| | 60 | 3 | 13 | | 4D | 3 | 13 |
| | 65 | 1 | 4 | | 4U | 3 | 13 |
| | 70 | 3 | 13 | | 6D | 1 | 4 |
| School speed limit (SSL), mph | 20 | 4 | 17 | School size[b] in number of students | 0–499 | 11 | 46 |
| | 25 | 1 | 4 | | 500–999 | 8 | 33 |
| | 30 | 2 | 8 | | 1,000–1,499 | 3 | 13 |
| | 35 | 17 | 71 | | 1,500–1,999 | 2 | 8 |
| Access density[c] (access) access points/mile | 0–3 | 2 | 8 | School driveway density[d] (num drive), driveways/mi | 0–2 | 8 | 33 |
| | 3–6 | 6 | 25 | | 2–4 | 6 | 25 |
| | 6–9 | 6 | 25 | | 6–8 | 3 | 13 |
| | 9–12 | 5 | 21 | | 8–10 | 2 | 8 |
| | 12–15 | 1 | 4 | | 10–12 | 2 | 8 |
| | 15–18 | 2 | 8 | | 12–14 | 2 | 8 |
| | >18 | 2 | 8 | | 18–20 | 1 | 4 |
| Area type (area) | Rural, mixed | 2 | 8 | School speed limit zone length[e] (SSZ), ft | 500–999 | 5 | 21 |
| | Rural, residential | 3 | 13 | | 1,000–1,499 | 6 | 25 |
| | Rural, undeveloped | 5 | 21 | | 1,500–1,999 | 3 | 13 |
| | Suburban, commercial | 4 | 17 | | 2,000–2,499 | 3 | 13 |
| | Suburban, mixed | 2 | 8 | | 2,500–2,999 | 3 | 13 |
| | Suburban, residential | 5 | 21 | | 3,000–3,499 | 1 | 4 |
| | Suburban, undeveloped | 3 | 13 | | 3,500–3,999 | 2 | 8 |
| | | | | | 4,000–4,499 | 1 | 4 |
| Crosswalk | No | 14 | 58 | Sidewalk | No | 16 | 67 |
| | Yes | 10 | 42 | | Yes | 8 | 33 |
| School type | Elementary | 10 | 42 | Data Collection Method | Counters | 5 | 21 |
| | Elementary and middle | 1 | 4 | | Laser | 12 | 50 |
| | Middle | 7 | 29 | | Both | 7 | 29 |
| | Middle and high | 1 | 4 | | | | |
| | High | 3 | 13 | | | | |
| | All | 2 | 8 | | | | |

NOTE: Freq. = frequency and TWLTL = two-way left turn lane.
[a]U = undivided, D = divided.
[b]Groups of 500 student increments used in this table, actual value used in regression analyses.
[c]Groups of 3 access points/mi increments used in this table, actual value used in regression analyses. Value determined by following equation: access density = 5,280 × (number access points)/(SSL zone length).
[d]Groups of 2 driveways/mi increments used in this table, actual value used in regression analyses. Value determined by following equation: school driveway density = 5,280 × (num drive)/(SSL zone length).
[e]Groups of 500-ft increments used in this table, actual value used in regression analyses.

If the vehicle turned in the study area, a note was recorded and the vehicle was removed from the data set.

To collect laser data, researchers identified locations near the schools with clear lines of sight upstream of the school zone. Generally, these locations were off the shoulder of the roadway, preferably behind trees or other roadside objects, to hide the observer from drivers' field of view. Observers parked vehicles in these locations and collected speeds from inside the vehicle. Observers used several means to minimize any effects of their presence on approaching drivers. First, observers used vehicles other than sedans to minimize the possibility that drivers would mistake the observer for an enforcement officer. Second, observers parked as far off the roadway as possible while still maintaining a clear line of sight through the study area. Third, observers did not raise their laser guns into position until the target vehicle had passed, eliminating the possibility of the driver recognizing the use of an active speed-measuring device.

For some sites, laser guns were not practical because of inadequate line-of-sight or unsuitable observation locations; for these sites, automated traffic data collectors were used. Because of their ability to collect data automatically, traffic counters collected data all day at each site at which they were installed, giving greater ability to compare peak and nonpeak periods.

## Data Analysis

For the counter data, postprocessing included removing data that were obviously a result of sensor error, data involving any vehicle traveling less than 5 mph (assumed to be a vehicle preparing to turn), and data on vehicles with less than a 5-s headway (assumed to be a non-free flow). For selected analyses, data from speed–distance profiles collected with laser were converted to spot-speed data and combined with the counter data. The laser data were also examined and evaluated as continuous speed–distance data. Sites were excluded from an analysis if the sample size of vehicle speeds was small (generally 30 or 40 vehicles). The number of data sets varied depending on whether spot-speed data or continuous speed–distance or speed–time data were used.

### Site Characteristics

Most sites had a posted school speed limit (SSL) of 35 mph, which is reflective of the rural and suburban nature of the sites selected. The sites with a regulatory speed limit greater than 55 mph required an intermediate speed limit to step down from the posted regulatory speed limit to the posted SSL. In Texas, this can be accomplished by using a regulatory speed limit or by using a school buffer zone. In the former, a 70-mph regulatory speed limit would be followed by a 55-mph regulatory speed limit and a 35-mph SSL. In the latter, which is unique to Texas, a 70-mph regulatory speed limit would be followed by a 55-mph SSL and a 35-mph SSL. Additional information on school buffer zones is contained elsewhere (*1, 4*); this paper will focus on speeds in the 35-mph-or-less school zones.

## SPEEDS IN SCHOOL ZONES

A goal of the speed data collection was to determine how school-zone characteristics affect vehicle speeds when the SSL is in effect (i.e., when the beacon is on). Table 1 lists the variables initially considered in the analysis. The question of how variables affect speeds in an active school zone was answered in three ways on the basis of three different data sets:

- Average speeds of vehicles in an active school zone obtained from 30 data sets (laser data sites and counter data sites, subdivided by direction of travel at six sites in which data were collected in both directions), number of observations ($n$) = 30.
- Spot-speed data obtained from individual vehicle speeds for vehicles in an active school zone, $n$ = 24,829.
- Speed–distance data obtained from individual vehicle speeds along the entire active school zone distance measured when beacon is on, $n$ = 59,966.

Before a determination of which variables affect speeds, general operating speed characteristics were explored. This section concludes with the results of the investigation into the location of the minimum speed in a school speed zone.

### Operating Speed Characteristics

Several variables were collected for use in evaluating the potential effects on operating speed in an active school zone. Observations of unique variables include the following:

- The average speed for sites with a crosswalk was significantly lower than the average speed for sites without a crosswalk.
- The average speed for sites without a sidewalk was significantly higher than the average speed for sites with a sidewalk.
- The average speed was higher for sites in rural areas. The lowest speeds recorded were in suburban residential areas, and the highest were in rural mixed and undeveloped locations.
- The lowest speeds were associated with speed zones of the shortest lengths, which were correlated with the lowest SSLs.
- Lower speeds were associated with a higher number of access points (for access density and school driveway density).

A pattern for number of lanes (num lanes) was not obvious. In summary, the data suggest that there are effects of crosswalk, area type, zone length, access or school driveway density, and sidewalk on average speeds when they are considered separately.

Figure 1*a* shows the recorded 85th percentile speed in active and inactive school speed zones, as compared with their respective SSLs. Figure 1*b* shows similar findings using average speed. The solid diagonal lines shown in each figure are the lines at which the 85th percentile (or average) speeds equal the posted speed. A simple check of the data points in Figure 1*a* reveals that almost all sites have 85th percentile speeds exceeding the posted SSL (shown with square symbols), and a comparison of the operating speed with the regulatory speed limit reveals that about half the sites had an 85th percentile speed slightly less than the regulatory speed limit for the site (shown with diamond symbols).

The following 85th percentile speeds by SSL value were identified:

| *SSL (mph)* | *85th Percentile Speed (mph)* |
|---|---|
| 20 | 26 |
| 25 | 30 |
| 30 | 37 |
| 35 | 43 |

### Analysis Based on Average Speeds

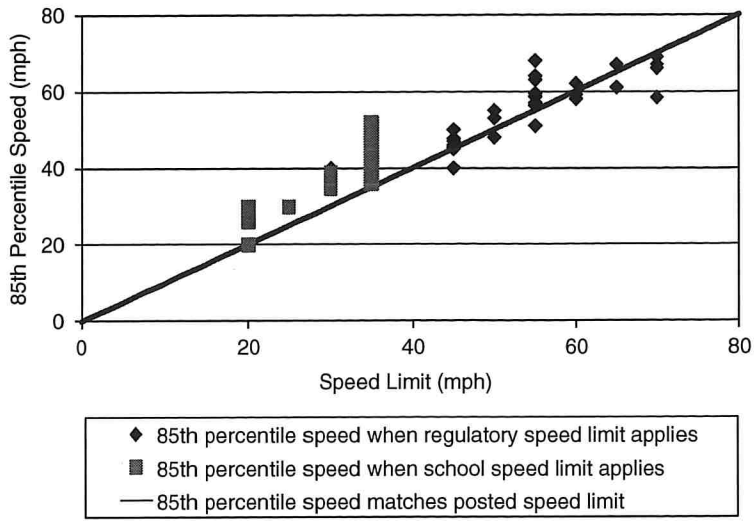
In the analysis using average speed, the dependent variable is the average speed for each site obtained as the average of the speeds measured in the school zone when the beacon was on. Multiple models were explored to find the most useful in assessing the effects of the school variables on average speeds. The effect of SSL dominated the effects of all other variables when it was included in a model. Therefore, researchers considered two types of models, one including SSL and one without.

Because some variables are correlated with one another, not all the school characteristic variables could be included in the same model simultaneously. For example, the sites with high SSL (35 mph) generally have longer school zone lengths compared with the sites with lower SSL. As a result, the effect of SSL on average speed is somewhat confounded with the effect of length of school zone. The following groups of variables were correlated:

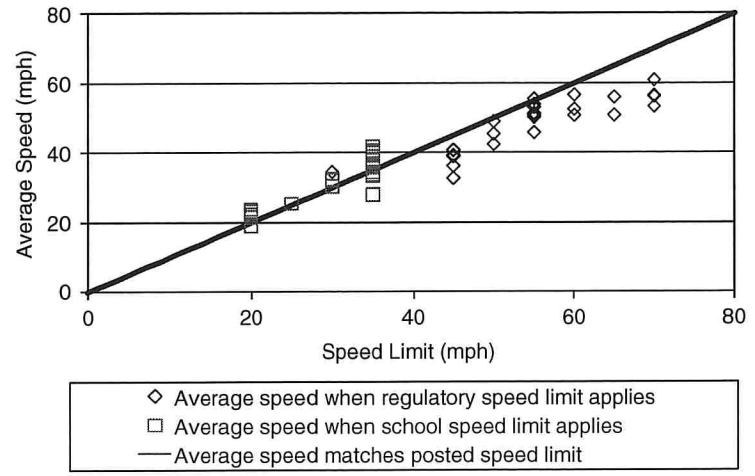
- Access density, school driveway density, and length of school zone;
- SSL and length of school zone;
- SSL and school size; and
- School size and length of school zone.

For the models that included it, SSL was the only significant variable. In general, these models had $R^2$ values of .92 ($R^2_{adj}$ = .84). The SSL values are associated with several other roadway characteristics; for example, the access or school driveway density would tend to be higher with lower SSLs. Because school speed limits are related to so many other variables, it is not surprising that it is the only variable significant in a model.

Among the models not including SSL as one of the independent variables, the model that contained area type, num lanes, school driveway density, school size, and school type seemed to be the best model. The effects of area type, num lanes, and school driveway density are statistically significant at the $\alpha$ = .05 level whereas school size was statistically significant at the $\alpha$ = .1 level. The effect of school driveway density on average speed is negative (regression coefficient = –0.6850), which agrees with other research that has found operating speeds to be lower as the number of driveways increases (*5*).




(a)




(b)

FIGURE 1 Observed speed versus speed limits: (*a*) 85th percentile speed and (*b*) average speed.

## Analysis Based on Spot-Speed Data

In the spot-speed analysis, the dependent variable is speed (measured as a spot speed for the counter data and reduced to a spot speed for the laser data) in a school zone when the beacon is on. Because the dependent variable is the individual vehicle speed, not the average speed, a car-related variable, relative minutes to start (or end) of school (rel min school), could be included as an additional independent variable in the analysis. The other variables considered for this analysis were area type, num lanes, and school size.

Researchers were also interested in determining whether the effect of rel min school is different for the four time periods corresponding to the time just before the start of school (a.m. before), the time just after school opens (a.m. during), the time just before school ends (p.m. during), and the time just after school is released (p.m. after).

There are two experimental units in this data set, sites (denoted as site num) and cars. It can be expected that the speed measurements from the same site will be more correlated (because of common site characteristics) than speed measurements from multiple sites. To address this within-site correlation, researchers analyzed the data by using a split-plot model, treating site num as a whole plot and each vehicle as a split plot. The school characteristic variables area, num lanes, and school size serve as whole-plot factors, and the variable rel min school serves as a split-plot factor. Sites were treated as a random effect (nested within area, num lanes, and school size) because researchers were interested in the characteristic variables describing the sites, such as area, num lanes, and school size, rather than the sites themselves.

Table 2 summarizes the findings from the analyses. For each period, all four variables included in the model (area, num lanes, school size, and rel min school) are statistically significant at $\alpha = .05$. The effect of rel min school varies depending on the time period. Before the start or end of school, the effect is negative, meaning that speeds decrease for those times closer to the start or end of school. Following the start or end of school, the effect of rel min school is positive, meaning that speeds increase as time increases. Effects are more pronounced in the morning periods, as shown in the magnitude of the rel min school coefficients.

TABLE 2 Results of Spot-Speed Analysis

| Analysis Parameter | a.m. Before | a.m. During | p.m. During | p.m. After |
|---|---|---|---|---|
| *R*-square adj. | .4273 | .4563 | .2620 | .2350 |
| Significant variables | Area<br>Num lanes<br>School size<br>Rel min school | Area<br>Num lanes<br>School size<br>Rel min school | Area<br>Num lanes<br>School size<br>Rel min school | Area<br>Num lanes<br>School size<br>Rel min school |
| Rel min school coefficient | −0.0515 | 0.0448 | −0.0189 | 0.0384 |
| Time (min) away from start or end of school resulting in 1-mph speed difference | 19 | 22 | 53 | 26 |
| Time (min) away from start or end of school resulting in 2-mph speed difference | 39 | 45 | 106 | 52 |

NOTE: Rel min school = relative minutes to start (or end) of school.

Although the rel min school variable is significant, is the finding of practical value? Table 2 lists the number of minutes away from the start or end of school when the estimated speed would represent a practical difference of 1 mph. If 1 mph is set as the practical difference (1 mph represents the typical accuracy of a laser gun), then a 1-mph speed difference would be experienced 19 min before the start of school or 22 min after the start of school. Stated another way, a morning active period of 41 min (19 min + 22 min) would result in the speeds being within 1 mph of the lowest speed during the morning active school zone. The effects of time for the afternoon period were not as intense. Speeds within a 1-mph range would be experienced for 53 min before the end of school to 26 min after the end of school. These findings do not address the magnitude of speed during the active school zone period—only the change in speeds as time changes. A key finding from this evaluation is that speeds are higher for greater time increments from the start or end of school.

## Analysis Based on Speed–Distance Data

Researchers were interested in the effect of the distance from the start of the school zone (rel dist SZ) on the speeds in an active school zone. Therefore, the original laser data were used so that rel dist SZ could be included in the analysis. Unlike the spot-speed data, there were multiple speed measurements for a single vehicle for the laser data because speed was measured continuously over a range of distances. Speed measurements corresponding to the same vehicle were, in general, highly correlated, and this within-vehicle correlation needed to be incorporated into the analysis in addition to the within-site correlation mentioned earlier in the analysis of the spot-speed data. A split-split-plot model treating site num as a whole plot, vehicle num as a split plot, and an individual speed measurement from a vehicle as a split-split plot was used for analyzing the original 59,966 speed measurements. The school characteristic variables area, num lanes, and school size serve as whole-plot factors, and the variables period and rel dist SZ serve as split-plot factor and a split-split-plot factor, respectively. Sites and vehicles were treated as random effects (sites are nested within area, num lanes, and school size; vehicles are nested within sites) because neither sites nor vehicles themselves were of interest.

The effects of area, num lanes, rel dist SZ, and period were statistically significant at $\alpha = .05$. The analysis showed that speeds increase as the relative distance within the school zone increases, by a factor of 0.0018583. Researchers examined the practical application of the speed–distance relationship, examining speed changes over varying school zone lengths. Speeds increase approximately 0.9 mph for every 500 ft in school zone length, or for every quarter-mile (1,320 ft) of school zone length, speeds can be expected to increase almost 2.5 mph.

## Minimum Speed in School Zone

Ideally, a speed–distance profile in a school speed zone when the school zone is active would show speeds at or below the posted speed limit consistently throughout the length of the zone; because of their consistency, those speeds would all be close or equal to the minimum recorded speed at that site. A review of the plots revealed that a bowl-like shape is present. An approach to examining the speed–distance profile is to identify the location of the minimum speed for each individual vehicle. Ideally this location would be at the location of the SSL sign.

Sites at which only counters were used to collect speed (five sites) or that included a buffer zone (seven sites) were not included in this evaluation. A review of the laser speed data for one of the remaining sites indicated that the minimum speed location may be in areas not covered by the laser readings; therefore this site was also removed. For the remaining 11 sites, the initial location of the minimum speed for each vehicle in its speed–distance profile was identified. For example, if the vehicle decelerated to a speed of 30 mph approximately 100 ft beyond the SSL sign and remained at the 30 mph for another 400 ft, the procedure identified 100 ft as the location of the minimum speed.

Cumulative distributions were generated for the sites included in this analysis as shown in Figure 2. The 0 distance point on the graph represents the location of the SSL sign. The graph shows that between 1% and 39% of the vehicles at a site were at the minimum speed at the start of the school zone. Minimum and maximum locations along with the 25th, 50th, and 75th percentile values were determined for the 11 sites and are listed in Table 3. Observations included the following:

- Minimum speeds occur over a large range of distances, ranging from approximately 500 ft upstream of the SSL sign to more than 2,000 ft downstream.
- Most minimum speeds occurred between 150 and 500 ft after the SSL sign.
- In general, minimum speeds occurred closer to the SSL sign for the shorter school zones. Figure 3 shows the plot of average location of minimum speed by school zone length. The plot shows an upward trend—longer school zone lengths are associated with

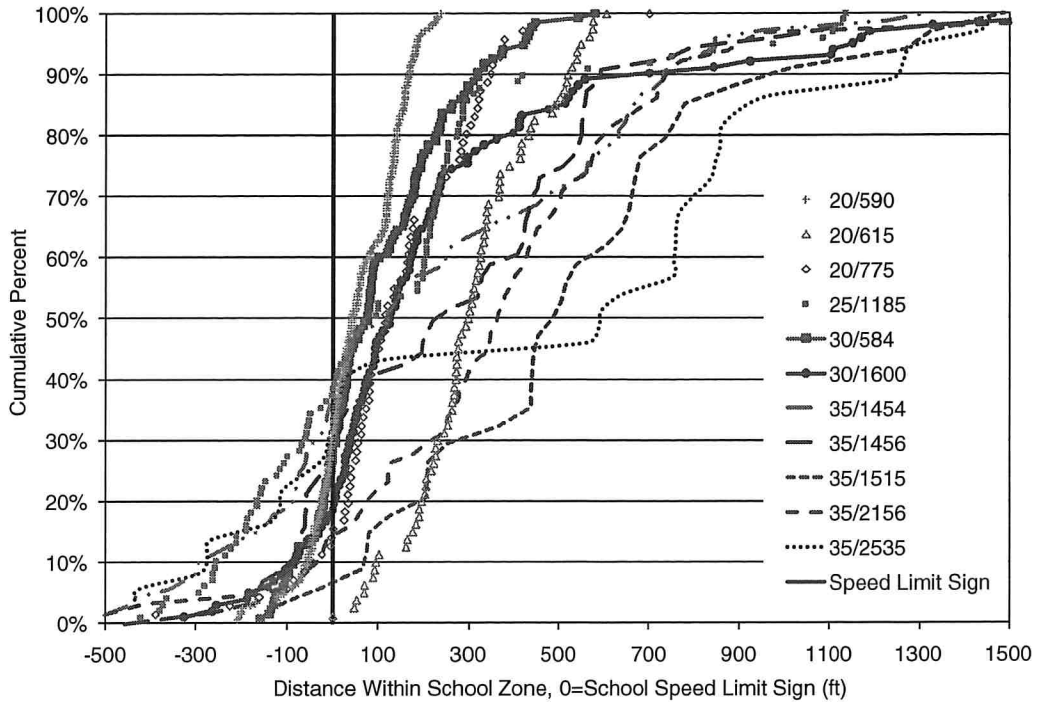



FIGURE 2 Cumulative distribution of individual vehicle minimum speed location in school zone (0 distance point is the location of the SSL sign).

larger distances for the minimum speed locations. Longer zones are associated with a minimum speed location a greater distance from the speed limit sign.

Organizing the findings by SSL, the average location for the minimum speed is 126 ft for the three 20-mph-zone sites, 128 ft for the 25-mph-zone site, 157 ft for the two 30-mph zones, and 338 ft for the five 35-mph zones.

## IMPLEMENTATION OF FIELD STUDY FINDINGS

Following are suggested traffic control devices and practices developed using the findings from this research.

### Distance Between Start of School Zone and Crosswalk

The 2006 *Texas Manual on Uniform Traffic Control Devices* (*3*) recommends 200 ft as the distance between the SSL (S5-1) sign and school driveway or marked crosswalk. The research team recommends that, rather than having a single value, the distance be sensitive to speed. Table 4 summarizes steps taken or examined to develop the recommended spacing between the SSL sign and the school driveway. Column 2 of Table 4 shows typical SSLs for given regulatory speed limits (shown in Column 1). Columns 3 and 4 show the corresponding 85th percentile speed measured in the field studies and the corresponding stopping sight distance (SSD), respectively.

TABLE 3 Location of Minimum Speed Relative to Start of School Zone

| School Speed Zone | | | | Distance from SSL Sign (ft) | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Site | Length (ft) | Speed Limit (mph) | Number of Vehicles | Average | Min. | 25th percentile | 50th percentile | 75th percentile | Max. |
| SA-2 | 590 | 20 | 213 | 52 | −207 | −9 | 41 | 132 | 237 |
| SA-3 | 615 | 20 | 80 | 309 | 0 | 222 | 300 | 396 | 605 |
| SA-1 | 775 | 20 | 71 | 144 | −389 | 44 | 116 | 272 | 701 |
| EL-1 | 1,185 | 25 | 99 | 128 | −423 | −112 | 94 | 247 | 1,138 |
| EL-3 | 584 | 30 | 135 | 102 | −159 | −3 | 77 | 193 | 579 |
| EL-2 | 1,600 | 30 | 102 | 230 | −327 | 28 | 125 | 286 | 1,602 |
| LE-1 | 1,454 | 35 | 73 | 219 | −503 | −60 | 119 | 562 | 1,315 |
| BR-2 | 1,456 | 35 | 56 | 244 | −456 | −27 | 217 | 468 | 1,137 |
| BR-1 | 1,515 | 35 | 34 | 526 | −117 | 223 | 495 | 676 | 1,487 |
| SW-2 | 2,156 | 35 | 65 | 385 | −497 | 125 | 366 | 571 | 1,986 |
| BR-3 | 2,535 | 35 | 37 | 458 | −430 | −20 | 595 | 847 | 1,457 |

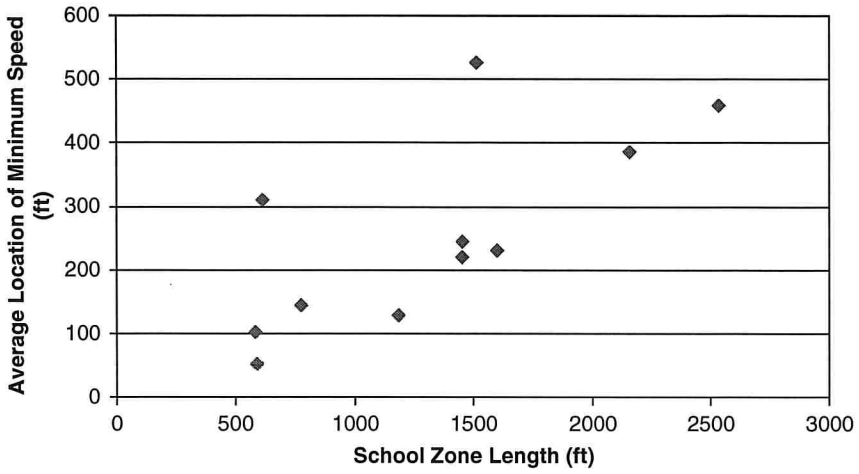



FIGURE 3 Minimum speed location compared with school zone length.

These distances were rounded to the suggested value in Column 5. Speeds in a school zone have a "bowl" shape with the minimum speed occurring between 100 ft and 500 ft beyond the SSL sign. If the speed pattern in the school zone (i.e., goal of having the lowest speed in a school zone be at the crosswalk) is considered with stopping sight distance, the recommended distance from the crosswalk or school driveway is 200 ft for 25 to 35 mph, 300 ft for 40 to 45 mph, and 400 ft for 50 mph and greater posted speed limits, as shown in the final column of Table 4. The location of the beginning or ending of a school speed zone should be based on engineering judgment with consideration of the pedestrian crossing locations rather than the exact location of the school property line or fence.

## SSL Zone Length

The suggested minimum school speed zone length is 400 ft, equal to two times the minimum distance between a SSL (S5-1) sign and a crosswalk or school driveway (200 ft) (see Table 4). The development of suggested speed zone lengths was more complex.

Other states' suggestions include the following:

- Massachusetts' amendment includes advice on speed zone length for rural (start of zone at least 850 ft in advance of grounds) and urban (either 500 ft or one block) (*6*).
- Michigan establishes school zones only for elementary and middle schools; they are generally defined as the portion of the road "1,000 ft from the property line of the school in each direction" (*7*).
- Washington states that the speed zone is 300 ft in either direction from the marked crosswalk (*8*).
- New York specifies a maximum length of 1,320 ft (*9*).
- Pennsylvania caps the distance at 1,600 ft (*10*).

The difference in speeds between the start of the school speed zone and the end of the school speed zone was determined through research (*1*). Speeds are approximately 2 mph higher for every 1,000 ft from the minimum point. For a SSL zone of 3,000 ft, the speed difference between the start of the zone and the end of the zone would be 6 mph.

Column 3 of Table 4 shows the 85th percentile speeds by SSL identified as part of the field studies. The 85th percentile speeds in school speed zones are between 5 and 8 mph greater than the SSL. Considering the research findings above, a 35-mph SSL zone with a 3,000-ft length would have an 85th percentile speed of 49 mph at the end of the SSL zone.

On the basis of these considerations, along with discussions with the project monitoring committee, a typical SSL zone length for higher-speed roadways of 1,000 ft or 0.2 mi (i.e., 500 ft or 0.1 mi either side of a driveway) was suggested.

## Other Suggested Treatments and Procedures

The research conducted as part of the Texas DOT project included discussions with the project monitoring committee, workshops at professional meetings, and presentations on the subject (*1*). Within

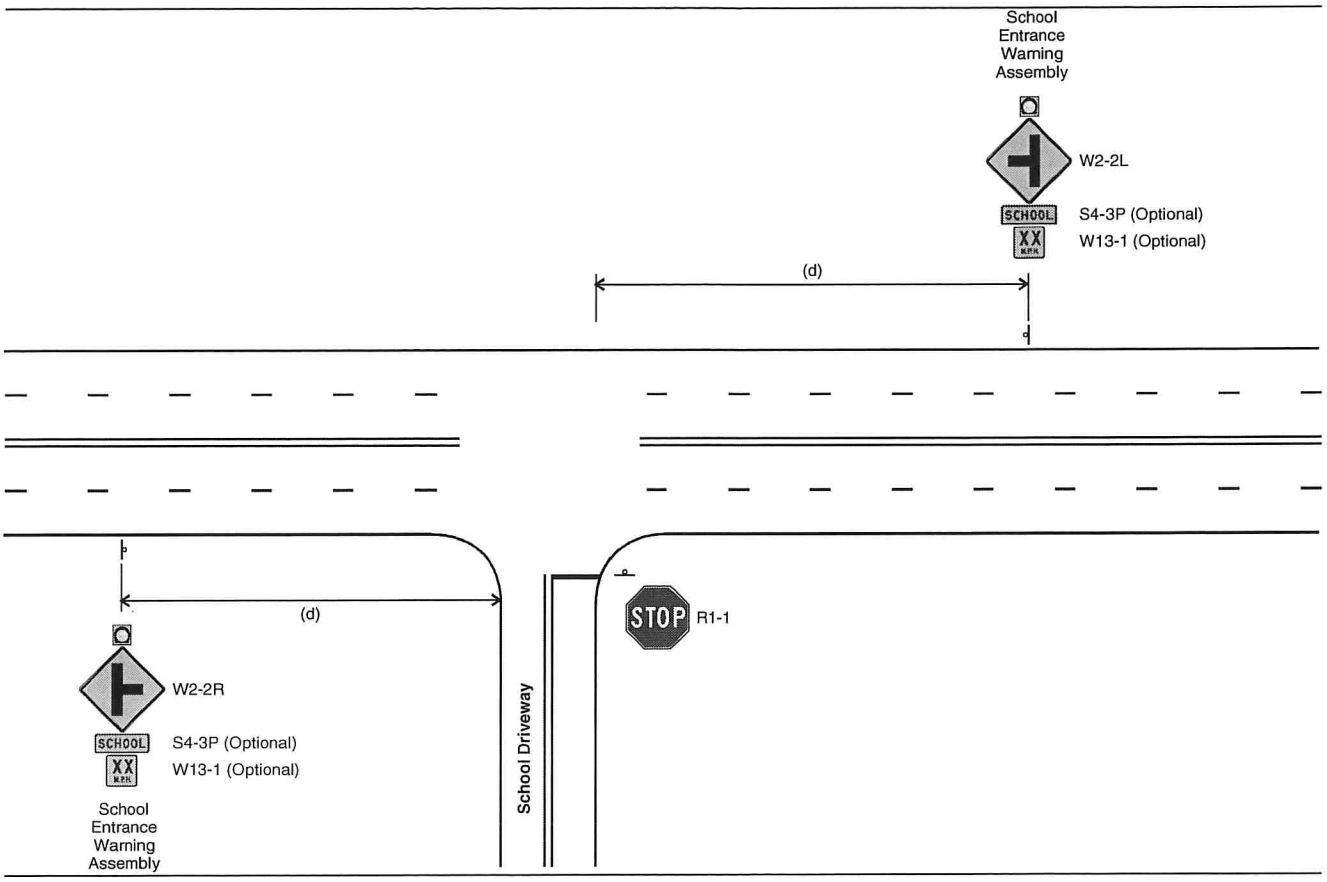
TABLE 4 Suggested Distances Between SSL Sign and School Driveway or Marked Crosswalk

| Regulatory Speed Limit (mph) (1) | Typical SSL (mph) (2) | Measured 85th Percentile Speed[a] (mph) (3) | SSD (ft) (Interpolated) (4) | SSD Distance, Rounded (ft) (5) | Minimum Speed Location[b] (ft) | | | Recommended Distance (ft) (7) |
|---|---|---|---|---|---|---|---|---|
| | | | | | (6a) | (6b) | (6c) | |
| 25–35 | 20 | 26 | 164 | 200 | 115 | 126 | 296 | 200 |
| 35–40 | 25 | 30 | 200 | 200 | 94 | 128 | 289 | 200 |
| 40–45 | 30 | 37 | 272 | 300 | 87 | 157 | 326 | 300 |
| 50–70 | 35 | 43 | 338 | 350 | 324 | 338 | 746 | 400 |

NOTE: SSD = stopping sight difference.
[a]Measured 85th percentile speed from field studies.
[b]Distance from school speed limit sign where minimum speed occurred, based on individual vehicles for field study sites with given school speed. Column 6a represents 50th percentile value, Column 6b represents average value, and Column 6c represents 85th percentile value.

those efforts several ideas for school traffic control were suggested, including the following:

- Use an assembly informing drivers of the presence of a school driveway. The assembly could include a suggested lower speed, but would not need to include a required speed change (as required with a SSL zone). Figure 4 shows the school entrance warning assembly developed in the Texas DOT project.
- Identify conditions to consider removal of a school speed zone. The following criteria are being used by the Texas DOT Dallas District and were incorporated into the Texas DOT guidelines (2):
  - If a traffic signal or all-way stop is installed at the entrance of a school, creating a controlled environment for both vehicle entrance and exit and a controlled pedestrian crossing;
  - If a school speed zone was previously established on the basis of vehicles stopped in a travel lane for left and right turns into the




Notes:

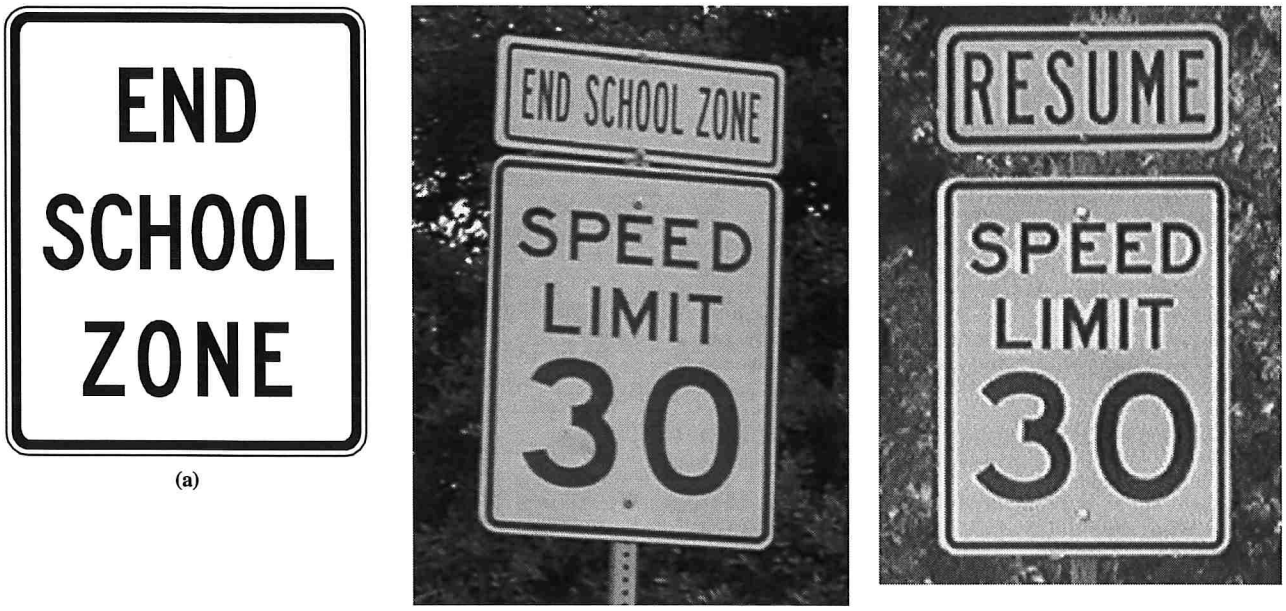
A School Entrance Warning Assembly (see figure above) is used to inform drivers of the presence of a school driveway. It should not be used if a school speed limit zone is present. The decision to use a School Entrance Warning Assembly should be based on engineering judgment. Conditions at the site could include the following:

- Crash records involving vehicles entering or leaving the school entrance during normal school hours indicate a need to advise drivers to reduce speed.
- The majority of students are transported to and from school by bus and/or private vehicles.
- No provisions are made for students to walk to and from school.
- There are no left- or right-turn lanes on the highway at the school driveway, or queue spillover caused by turning vehicles is present, or measures to address the spillover have not corrected the situation.
- The entrance is not controlled by traffic signals.

A school entrance warning advisory plaque can be included at up to 15 mph below the normal posted speed limit.

| Posted or 85th Percentile Speed (mph) | 25 | 30 | 35 | 40 | 45 | 50 | 55 | 60 | 65 | 70 |
|---|---|---|---|---|---|---|---|---|---|---|
| Distance (d) Between School Entrance Warning Assembly to School Driveway (ft) | 225 | 325 | 450 | 550 | 650 | 750 | 850 | 950 | 1100 | 1200 |

FIGURE 4 Suggested guidance for school entrance warning assembly (1).




(a)

(b)

FIGURE 5 Examples of methods used to end school zone.

school, and left- and right-turn bays have been added to adequately separate stopped vehicles from through traffic;

– If a school speed zone was previously established on the basis of limited sight distance on the highway approaching the entrance to the school and a highway improvement project has removed the sight-distance restriction; and

– If pedestrian patterns have changed as a result of changes in walking behavior or changes in bus ridership.

• Use alternatives to the use of the "End School Zone" sign such as the following shown in Figure 5:

– "Resume" plaque or

– "End School Zone" plaque.

## CONCLUSIONS

The purpose of a recent Texas DOT project, *Comprehensive Guide to Traffic Control Near Schools,* was to examine traffic control treatments used near schools, especially those associated with reduced-speed school zones located on higher-speed roadways (*1*). The findings were used in developing suggested guidelines for traffic control devices for school speed zones. These guidelines, titled *Guidelines for Traffic Control for School Areas,* are available on the Internet (*2*). They are intended to serve as a supplement to the *Texas Manual on Uniform Traffic Control Devices* (*3*) and the Texas manual *Procedures for Establishing Speed Zones* (*4*). This paper focuses on the findings from the field studies and presents suggestions concerning school traffic control devices. Key field findings include the following:

• For most sites, drivers are in better compliance with the regulatory speed limit than with the SSL. When the regulatory speed limit was in effect, about half the sites had a compliance rate of 60% or better. When the SSL was active, about half the sites had a compliance rate of less than 50%, with some lower than 10%. Compliance does not appear to be a function of the SSL; each speed limit had a broad range of compliance values (9% to 75% for 20-mph speed limit and 12% to 94% for 35-mph speed limit).

• In the regression analysis that identified variables affecting operating speed in an active school zone, SSL dominated all other variables.

• The effects of area type, num lanes, and school driveway density on average operating speed are statistically significant at the $\alpha = .05$ level when SSL is not in the model. The effect of school driveway density on average speed is negative, which agrees with other research that found operating speeds to be lower as the number of driveways increases.

• Speeds are higher for greater time increments from the start or end of school.

• Speeds increase as the relative distance in the school zone increases, by a factor of 0.0018583. The practical application is that speeds increase approximately 0.9 mph for every 500 ft in school zone length. Thus, longer school zones do not result in lower speeds for a longer distance.

• For individual drivers, minimum speeds occurred over a large range of distances (from 500 ft upstream of the SSL sign to more than 2,000 ft downstream), but most minimum speeds occurred between 150 and 500 ft after the SSL sign. In general, minimum speeds occurred closer to the SSL sign for the shorter school zones.

• The measured 85th percentile speed was 26 mph for the 20-mph SSL, 30 mph for the 25-mph SSL, 37 mph for 30-mph SSL, and 43 mph for the 35-mph SSL.

This paper also presents suggestions on a school entrance warning assembly, criteria for the removal of a school speed zone, and alternatives for ending a school speed zone.

## ACKNOWLEDGMENTS

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*The Traffic Control Devices Committee peer-reviewed this paper.*

# Perceptual Measures to Influence Operating Speeds and Reduce Crashes at Rural Intersections

## Driving Simulator Experiment

Alfonso Montella, Massimo Aria, Antonio D'Ambrosio, Francesco Galante, Filomena Mauriello, and Mariano Pernetti

**The aim of this paper is to investigate, by means of a dynamic driving simulator experiment, the behavior of road users at rural intersections in relation to perceptual measures designed for increasing hazard detection. In the experiment 10 configurations of tangents were tested: Alt1, base tangent; Alt2, four-leg base intersection; Alt3, intersection with reduced sight distance; and Alt4 through Alt10, intersections with perceptual treatments. The Virtual Environment for Road Safety high-fidelity dynamic-driving simulator, operating at the Technology Environment Safety Transport Road Safety Laboratory located in Naples, Italy, was used. Analysis of the results used two approaches: (*a*) explorative description of data by cluster analysis and (*b*) inferential procedures about population using statistical tests. Results showed that the speed behavior in the tangents was significantly affected by the presence of the intersections and by the perceptual treatments. Intersections without perceptual treatments significantly affected driver speeds in the 250 m preceding the intersection. Perceptual treatments helped the driver to detect the intersection earlier and to slow down. Dragon teeth markings, colored intersection area, and raised median island performed better than the other perceptual treatments. They produced significant average speed reduction in the 150 m preceding the intersection ranging between 16 km/h and 23 km/h. Study results support real-world implementation of perceptual measures in rural intersections because they are low-cost, fast implementation measures with a high potential to be cost-effective.**

In transportation, human error is found to be a significant contributing factor in most crashes. Road users of all types make frequent mistakes because of the physical, perceptual, and cognitive limitations of human beings. Road users make errors of judgment concerning, for example, appropriate speeds to approach intersections. In some instances, it is the road environment that gives the wrong cues to the drivers suggesting operating speeds that are incorrect in relation to the objective crash risk.

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Speeding inconsistently with the road environment is considered to be a major crash contributing factor. Estimates are that in about one-third of fatal crashes excessive speeds are involved (*1*). Most road agencies attempt to achieve the right operating speed among drivers by imposing speed limits. Unfortunately several studies have shown that the speed limit is very much violated and, at best, serves only as a guide to drivers (*2–5*). The major reason for not obeying speed limits is that the limits are not reflective of roadway conditions and most drivers feel that the limits are lower than what the roadway can accommodate (*6*).

Crash occurrence does not depend only on the laws of physics. It depends also on driver adaptation to the road environment. Driver response in regard to speed is a common and important adaptation with safety consequences. Perceptual cues to traffic hazards can help the driver to detect them earlier, to slow down, and ultimately to save lives (*7*). Perceptual techniques, albeit not widely used, have low implementation cost and few constraints (*8*).

The aim of this paper is to investigate, by means of dynamic driving simulator studies, road users' behavior at rural intersections in relation to different perceptual measures designed for increasing hazard detection.

## PREVIOUS STUDIES

Rural highways are generally associated with long tangents and infrequent intersections, which can cause drivers to become inattentive (*9*) and underestimate speed (*10*).

In the literature many operating speed prediction models have been developed, but only marginal attention has been dedicated to the effect of the intersections on the speed profile. According to the Italian geometric design standard, the speed profile is not affected by intersections (*11*). As a result, very long auxiliary lanes and large sight distance are required. AASHTO's "Green Book" assumes that an approaching turning vehicle can decelerate comfortably up to 15 km/h in a through lane before entering the left-turn lane (*12*).

The reduction of vehicle speeds on the approaches to intersections was the focus of NCHRP Project 3-74 (*13, 14*). Speed effects were evaluated with reference to several different treatments. Effective treatments were transverse pavement markings, transverse rumble strips, combination of individual treatments such as lane narrowing and lateral pavement marking, or splitter island on each approach. Rumble strips were identified as having the greatest need for evaluation on the basis of their potential effectiveness.

Thompson et al. studied the effects of transverse rumble strips on vehicle speeds on approaches to intersections (*9*). Statistically significant reductions in approach speeds in the range between 3 and 5 km/h were observed.

As part of the FHWA effort to reduce intersection crashes, two concepts have been identified: (*a*) lane narrowing by means of rumble strips on outside shoulders and in a painted yellow median island on major road approaches and (*b*) channelizing separator islands on side road approaches (*15, 16*). Implementation of the lane narrowing concept produced statistically significant (at the 95% confidence level) average speed reductions equal to 5.6 km/h.

The effectiveness of traffic calming treatments has been assessed also by means of driving simulator experiments. Indeed, driving simulators have a potential to explain interaction between drivers and roadway surroundings and explore effective countermeasures (*17*). In support of this concept, many validation studies related to driving speed behavior have shown that drivers' speed performance in driving simulators is similar to that measured in the real world or in real instrumented cars.

Various traffic calming treatments, in a rural single-carriageway environment, were simulated in the Leeds driving simulator (*18*). Traffic calming measures were simulated in entrances to sharp bends and rural villages. Significant reductions in speed and in speed variance were observed in both situations.

Godley et al. used an advanced driving simulator for the evaluation of speeding countermeasures (*19*). Participants drove on roads that contained transverse rumble strips at three sites: the approaches to stop sign intersection, right curve, and left curve. Each treatment site with rumble strips paired off with an equivalent control site. Twenty-four participants drove an instrumented car and 20 participants drove the simulator in two separate experiments using mature drivers. Participants drove on roads that contained transverse rumble strips at three sites, as well as three equivalent control sites. At the treatment sites, speeds were slower than at the control sites in both experiments. A close correlation between speeds in the simulator and in the real driving situations was found.

Katz et al. investigated several alternative peripheral transverse bars designs in the highway driving simulator at Turner-Fairbank Highway Research Center (*20*). Four curves and two tangent sections were treated with peripheral transverse bars including four different design patterns. First, the participant drove the roadway under baseline conditions. Second, the participant drove the roadway with one set of experimental treatments. Results showed that treatments were effective in reducing speeds, but there were no overall significant differences between the different treatments.

## DESIGN ALTERNATIVES

In the experiment, a two-lane rural highway with lane width equal to 3.50 m and shoulder width equal to 1.25 m was simulated. The experimental route consisted of the succession of 20 tangents with length equal to 1,000 m and curves with radius equal to 400 m and deflection angle equal to 35°. The tangent-to-curve transition was carried out by spiral curves with length equal to 55 m, which corresponds to 2.0 s at 100 km/h. During the route the drivers ran 10 different tangent configurations (see Figure 1):

Alt1. Base tangent without intersections. Two-lane rural highway (lane width = 3.50 m, shoulder width = 1.25 m). Length equal to 1,000 m.

Alt2. Base intersection (see Figure 1*a*). Four-leg intersection located in the middle of the tangent. No skew angle (90° intersection angles). No auxiliary lanes. Intersection triangular warning sign on the major road located 150 m before the intersection. Yield controlled minor road. Minor road with lane width equal to 3.00 m and shoulder width equal to 0.50 m. Sight distance 20 m before the yield line equal to 330 m.

Alt3. Reduced sight distance. Stop controlled minor road. Sight distance 3 m before the stop line equal to 100 m.

Alt4. Optical speed bars (see Figure 1*b*) located on the main road 150 m and 80 m before the intersection. Each set consists of 6 bars, with length equal to 3.25 m, width increasing in the traveling direction (0.20, 0.30, 0.40, 0.50, 0.60, and 0.70 m), and spacing decreasing in the traveling direction (1.25, 1.15, 1.05, 0.95, and 0.85 m).

Alt5. Transverse rumble strips (see Figure 1*c*) located on the main road 150 m and 80 m before the intersection. Each set consists of nine raised rumble strips, with length equal to 3.25 m, width equal to 0.12 m, and spacing equal to 0.50 m.

Alt6. Peripheral transverse bars (see Figure 1*d*). One set of transverse rumble strips located on the main road 150 m before the intersection. Twenty-two couples of peripheral transverse bars located after the rumble strips. Bars are 0.44 m long (perpendicular to roadway edge) and 0.30 m wide (parallel to roadway edge). Spacing between each pair of bars determined using a constant deceleration rate, equal to 0.80 m/s$^2$, based on beginning speed equal to 90 km/h. Total length of the device equal to 121.50 m.

Alt7. Dragon teeth markings (see Figure 1*e*). Two sets of transverse rumble strips located on the main road 150 m and 80 m before the intersection. Dragon teeth markings located after the second set of rumble strips. Two series (one per direction) of 50 dragon teeth markings, with constant base and spacing, equal respectively to 0.30 m and 1.00 m, and increasing height (1 cm per meter): the first one 0.30 m, the 50th one 0.79 m.

Alt8. Colored intersection area (see Figure 1*f*). Two sets of transverse rumble strips located on the main road 150 m and 80 m before the intersection. Strip of red surface with printed bituminous concrete appearing as brick pavers in the intersection area, producing a slight vibration to the vehicles crossing the brick area.

Alt9. Painted median island (see Figure 1*g*). One set of transverse rumble strips located on the main road 150 m before the intersection. Symmetrical painted median island beginning after the rumble strips. Length of the island equal to 134.50 m. Width of the island equal to 1.50 m. Width of the lanes narrowed to 2.75 m.

Alt10. Raised median island (see Figure 1*h*). One set of transverse rumble strips located on the main road 150 m before the intersection. Raised island with mountable curbs (w = 0.10 m, h = 0.05 m) separated from the lanes by two shoulders 0.50 m wide. Nose of the raised island located 33 m after the beginning of the painted island. Yellow reflective obstacle delineator, coupled with the sign "Passage Allowed to the Right" installed in the nose of the island.

## DRIVING SIMULATOR EXPERIMENT

### Apparatus

The Virtual Environment for Road sAfety (VERA) dynamic-driving simulator, operating at the Technology Environment Safety Transport (TEST) Road Safety Laboratory (see Figure 2) located in Naples, Italy, was used.

Three flat screens (3.00 × 4.00 m) are fixed at the simulation room floor to surround the motion platform. The visual scene is projected

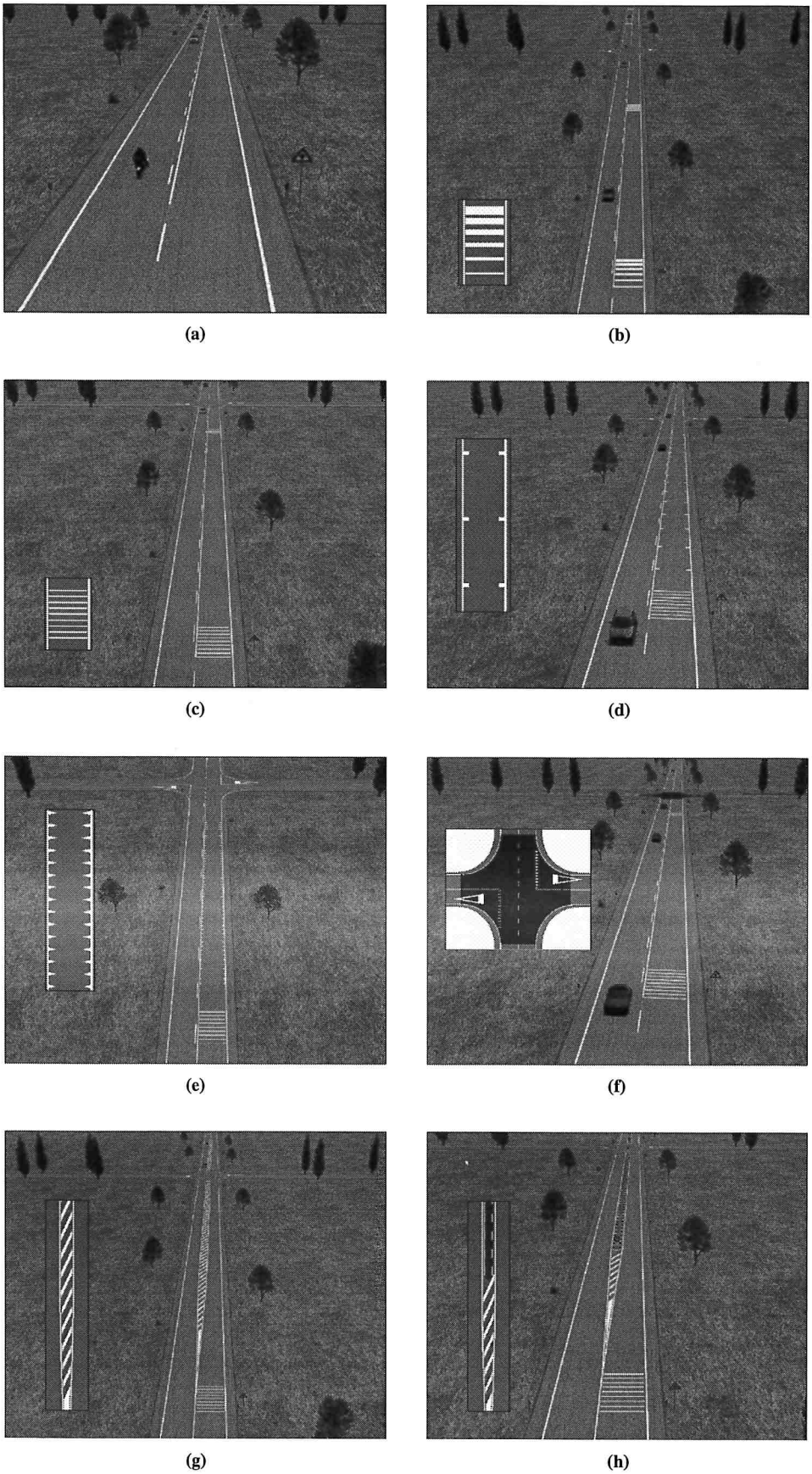

FIGURE 1 Design alternatives: (*a*) base intersection, (*b*) optical speed bars, (*c*) transverse rumble strips, (*d*) peripheral speed bars, (*e*) dragon teeth markings, (*f*) colored intersection area, (*g*) painted median island, and (*h*) raised median island.

(a) (b)

FIGURE 2 VERA driving simulator.

to a high-resolution, three-channel 180° × 50° forward field of view with rear and side mirror views replaced by 6.5-in. LCD monitors. The visual system allows a resolution equal to 1,400 × 1,050 for each channel and a refresh rate equal to 60 Hz. To minimize the flickering effect and enhance the image quality of the driving scenarios, an 8X anti-aliasing and an 8X anisotropic filtering are enabled. The cockpit is half of a real Citroen C2. The audio system can reproduce various sounds that can normally be heard while driving, including the rolling, engine, and exhaust noise produced by the driving vehicle as well as the surrounding sound field from other vehicles. Feedback is provided by a force feedback system (SENSO-Wheel SD-LC) on the steering and a 6 degrees of freedom electric motion platform. The torque feedback at the steering wheel is provided via a motor fixed at the end of the steering column. The motion system consists of a hexapod with six electric actuators able to reproduce most of the accelerations that real car occupants feel, in particular those arising from turning and braking maneuvers and from dynamic interaction between the vehicle and the pavement surface unevenness (*21, 22*). The driving simulation software used in VERA is SCANeR II r2.22 from Oktal company.

The use of driving simulators offers a number of positive elements, such as experimental control, efficiency, safety, and ease of data collection, but simulators must have appropriate validity to be useful as research tools (*23, 24*). To validate the VERA driving simulator speed data, comparison between real-world and driving simulator speed data was done in a previous study (*25*). The comparison showed that there were no significant differences between the real and the simulated speed samples.

### Procedure

Thirty participants were recruited for the experiment on the basis of a preliminary questionnaire. On their arrival in the laboratory, each participant was briefed on the requirements of the experiment and all read and signed an informed consent document. Consistent with previous studies, in this study each subject drove a learning route for 10 min. After a rest, each participant drove a route composed of the 10 alternatives twice. To test the behavior of drivers familiar with the treatments, the first run was used as the adaptation route and the second run was used as the test route. To prevent confounding errors, the sequence in which subjects encountered each alternative through the route was balanced. Overall, drivers encountered each alternative the same number of times in each driving order; that is, in both runs any alternative (from Alt1 to Alt10) was encountered in any sequential order (from the first to the 10th) by three drivers.

Seven subjects, five women and two men, exhibited simulator sickness and did not complete the experiment. Twenty-three participants, 13 men and 10 women between 23 and 55 years of age (mean = 35.2 years; standard deviation = 9.3 years) and having valid Italian driving licenses for more than 4 years (mean = 15.5 years; standard deviation = 10.0 years), completed the experiment.

## RESULTS AND DISCUSSION

Results were analyzed using two different approaches: (*a*) explorative description of data by cluster analysis and (*b*) inferential procedures about population using statistical tests. Cluster analysis was carried out to test whether the drivers' speed behavior in the different design alternatives was substantially different. Statistical tests were performed to verify whether speeds in specific sections were significantly different. Cluster analysis looked at speed profiles, whereas statistical tests looked at speed data at specific points.

### Cluster Analysis

Cluster analysis is a multivariate statistical methodology aimed at partitioning $N$ observations into $K$ disjoint groups in such a way that they

are maximally internally homogeneous and externally heterogeneous (*26*). In the definition of these groups, a distance measure must be defined. In this case, a nonhierarchical cluster analysis was performed, namely the number $K$ of the groups is a priori defined.

Because the nature of the speed data can be considered as time/spatial series, cluster time-/spatial-series methods were used. Clustering time-/spatial-series data can be defined as an optimization problem, for which the goal is to partition $N$ time/spatial series (instead of $N$ observations) into $K$ disjoint subsets. As distance measure, the Pearson's correlation coefficient was adopted because it is invariant to the scale and to the mean of the series.

Consistent with previous studies, clustering was performed by the $K$-means algorithm (*27*). The $K$-means algorithm works as follows: (*a*) Step 1, the number of clusters $K$ is chosen; (*b*) Step 2, random $N$ initial means are selected as starting points for the clusters; (*c*) Step 3, for each series, the similarity measure with each mean series is computed; each series is assigned to the cluster whose mean series has the highest similarity with the time series; (*d*) Step 4, means are updated; and (*e*) Step 5, Step 3 is repeated until no reallocation in the clusters occurs after the updating step or a maximum number $p$ of iterations is performed.

After cluster analysis, a silhouette analysis was performed. Silhouette refers to a method of interpretation and validation of clusters of data. The technique provides a concise graphical representation of how well each object lies within its cluster. It is assumed that the data have been clustered into $K$ clusters. For each object $I$, let $a_{(i)}$ be the average distance of $i$ from all other data in the same cluster that measures how well matched $i$ is to the cluster to which it is assigned (the smaller the value, the better the matching). Let $b_{(i)}$ be the minimum average distance of $i$ from the data of other single clusters. Define

$$s_{(i)} = \frac{b_{(i)} - a_{(i)}}{\max\{b_{(i)}, a_{(i)}\}} \quad (1)$$

with the silhouette value $s_{(i)}$ being bound between −1 and 1.

The average $s_{(i)}$ of a cluster is a measure of how all the data in the cluster are tightly grouped. Silhouette plots and averages are a powerful tool for determining the natural number of clusters in a data set. Values of $s_{(i)}$ greater than 0.6 are considered acceptable; values greater than 0.8 are considered very good (*26*).

Each driving alternative was divided into three segments: (*a*) from −250 m to −150 m (beginning of the perceptual treatments), (*b*) from −150 m to −75 m, and (*c*) from −75 m to 0 (tangent/intersection center). For any segment, cluster analysis with $K$ ranging between 2 and 6 was performed. Average silhouette values (see Table 1) were always smaller than 0.6 for $K > 5$, thus showing ineffective cluster structure for $K > 5$. The greatest number of partitions with values of $s_{(i)}$ greater than 0.8 was selected as the most effective: $K = 3$ in Segment 1 (−250 m, −150 m); $K = 4$ in Segment 2 (−150 m, −75 m), and $K = 5$ in Segment 3 (−75 m, 0).

TABLE 1 Silhouette Mean Values for Cluster Analysis

| Segment (m) | $K=2$ | $K=3$ | $K=4$ | $K=5$ | $K=6$ |
|---|---|---|---|---|---|
| −250 to −150 | 0.966 | **0.875** | 0.681 | 0.612 | 0.487 |
| −150 to −75 | 0.920 | 0.941 | **0.872** | 0.777 | 0.579 |
| −75 to 0 | 0.824 | 0.913 | 0.794 | **0.820** | 0.532 |

NOTE: Boldface indicates most effective number of partitions.

In the segment from −250 m to −150 m, the clusters were (see Table 2 and Figure 3) (*a*) the base tangent without intersections (Alt1), (*b*) the intersections with the perceptual treatments (Alt4-5-6-7-8-9-10), and (*c*) the intersections without perceptual treatments (Alt2-3).

In the segment from −150 m to −75 m, the clusters were (see Table 2 and Figure 4) (*a*) the perceptual treatments except the transverse rumble strips (Alt4-6-7-8-9-10), (*b*) base tangent (Alt1), (*c*) transverse rumble strips (Alt5), and (*d*) intersections without perceptual treatments (Alt2-3).

In the segment from −75 m to 0, the clusters were (see Table 2 and Figure 5) (*a*) the tangent without intersections (Alt1); (*b*) optical speed bars, transverse rumble strips, peripheral speed bars, and painted median island (Alt4-5-6-9); (*c*) colored intersection area (Alt8); (*d*) intersections without perceptual treatments (Alt2-3); and (*e*) dragon teeth markings and raised median island (Alt7-10).

## Statistical Tests

Speed data in the following sections were analyzed (see Tables 3 and 4): −500 m, −250 m, −150 m (beginning of the perceptual treatments), −75 m, 0 (center of tangent/intersection).

The normality assumption of the speed data distribution was verified using the Shapiro–Wilk, Anderson–Darling, Lilliefors, and Jarque–Bera tests. The homoscedasticity assumption of the speed data distribution was verified using the Fisher test. According to

TABLE 2 Cluster Description

| Segment (m) | $K$ | Cluster 1 | Cluster 2 | Cluster 3 | Cluster 4 | Cluster 5 |
|---|---|---|---|---|---|---|
| −250 to −150 | 2 | Alt1-2-3 | Alt4-5-6-7-8-9-10 | | | |
| | **3** | **Alt1** | **Alt4-5-6-7-8-9-10** | **Alt2-3** | | |
| | 4 | Alt4-6-7-8-9-10 | Alt5 | Alt1-2 | Alt3 | |
| | 5 | Alt1-2 | Alt3 | Alt4-6-7-8-9 | Alt5 | Alt10 |
| −150 to −75 | 2 | Alt1-2-3 | Alt4-5-6-7-8-9-10 | | | |
| | 3 | Alt2-3 | Alt4-5-6-7-8-9-10 | Alt 1 | | |
| | **4** | **Alt4-6-7-8-9-10** | **Alt1** | **Alt5** | **Alt2-3** | |
| | 5 | Alt5 | Alt2-3 | Alt7-8-10 | Alt1 | Alt 4-6-9 |
| −75 to 0 | 2 | Alt4-5-6-7-8-9-10 | Alt1-2-3 | | | |
| | 3 | Alt2-3 | Alt4-5-6-7-8-9-10 | Alt1 | | |
| | 4 | Alt 4,6,7,8,9-10 | Alt1 | Alt5 | Alt2-3 | |
| | **5** | **Alt1** | **Alt4-5-6-9** | **Alt8** | **Alt2-3** | **Alt7-10** |

NOTE: Boldface indicates the alternatives belonging to the most effective clusters.

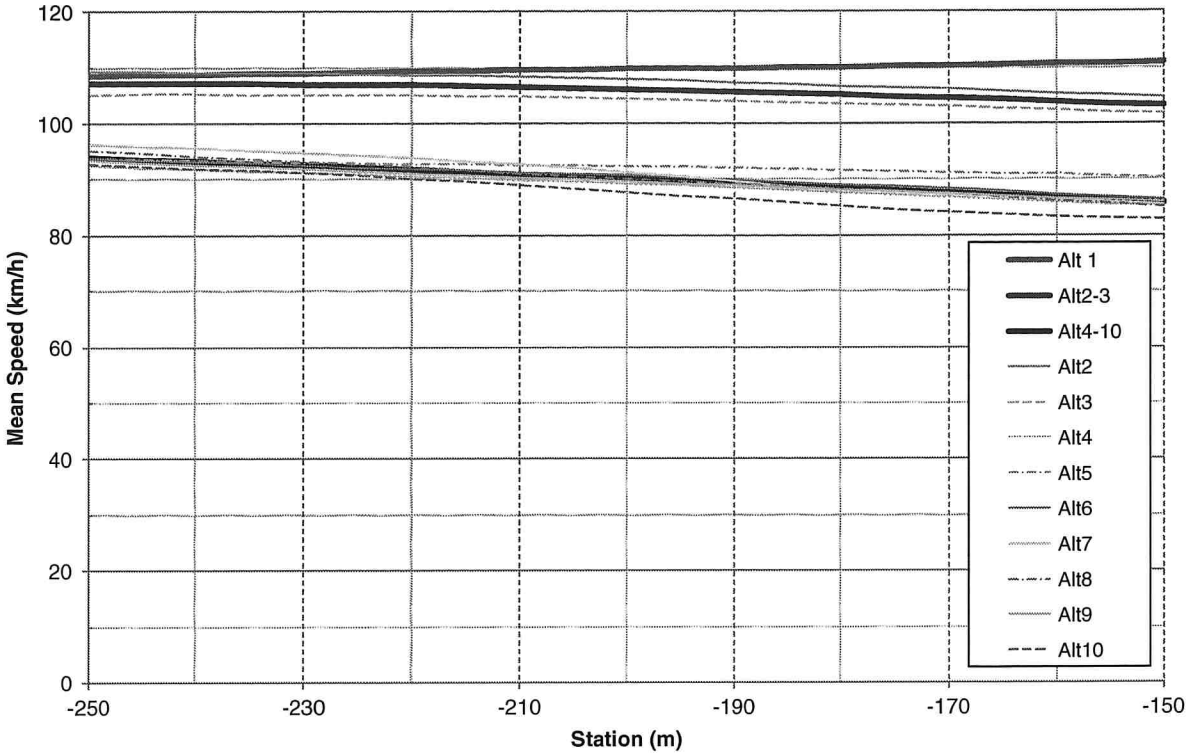


FIGURE 3 Speed profile and cluster results from −250 to −150 m ($K = 3$).

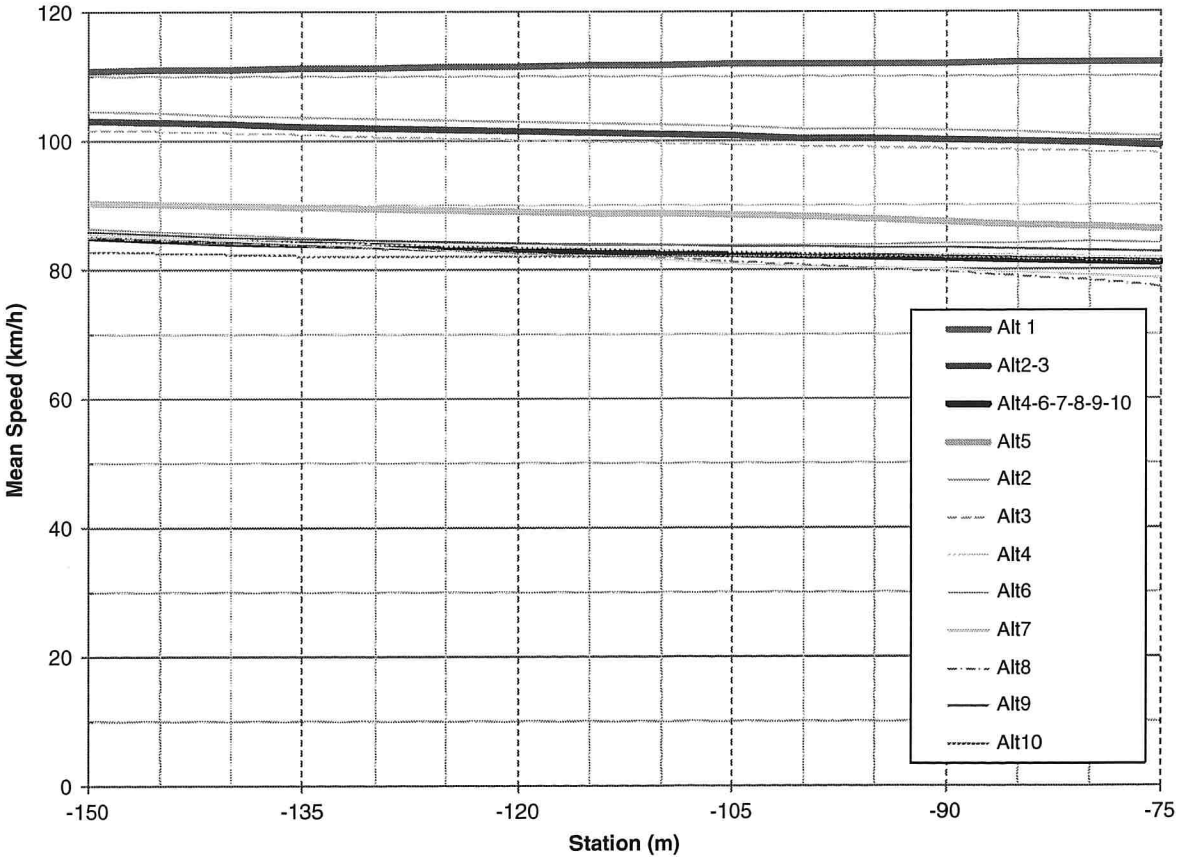


FIGURE 4 Speed profile and cluster results from −150 to −75 m ($K = 4$).

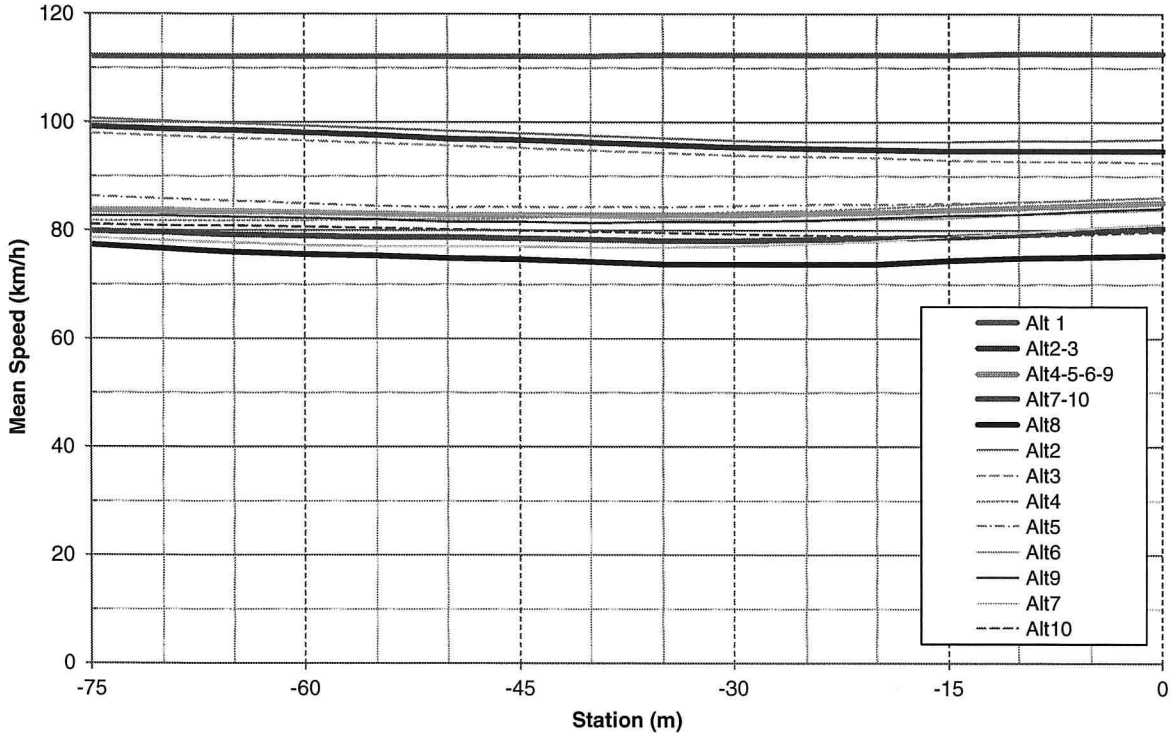


FIGURE 5 Speed profile and cluster results from −75 m to 0 ($K$ = 5).

the tests, the normality and homoscedasticity assumptions cannot be rejected at the 5% level of significance.

Given the normality and homoscedasticity of the data, at each section the speed data in the different alternatives were compared by parametric tests. To evaluate the presence of an overall effect, the analysis of variance (ANOVA) test was performed at each section considering all the alternatives (see Table 4). Results of the ANOVA tests showed that an overall effect was present in all sections (−250 m, −150 m, −75 m, 0) except the −500-m section. The latter result (i.e., the effect of the intersection on drivers' speed is not significant 500 m before the intersection) is consistent with previous findings in the literature. To assess whether the mean speeds between the alternatives were significantly different, Student's two-tailed paired $t$-tests were carried out (see Table 4).

In the section −500 m, mean speeds ranged between 99 km/h and 104 km/h, and most of the differences between the alternatives were not significant. Overall, intersections did not affect driver speeds.

In the section −250 m, the mean speed differences between the base tangent (Alt1) and the tangents with intersections without perceptual treatments (Alt2-3) were not significant. In all the alternatives with perceptual treatments (Alt4-10), mean speeds were significantly smaller than in the base tangent and in the intersections without treatments ($p < .001$), with differences in relation to the base intersection (Alt2) ranging between 13 km/h and 17 km/h. Perceptual treatments did not show significant differences.

In the section −150 m, the mean speed differences between the base tangent (Alt1) and the intersections without perceptual treatments (Alt2-3) were significant ($p < .05$). Intersection sight distance (Alt2

TABLE 3 Comparison of Speed Data in Different Alternatives

| | Mean Speed (km/h) | | | | | Standard Deviation (km/h) | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| Alternative | −500 m | −250 m | −150 m | −75 m | 0 | −500 m | −250 m | −150 m | −75 m | 0 |
| Alt1 | 101.40 | 108.50 | 110.86 | 112.21 | 112.68 | 16.05 | 16.72 | 16.91 | 17.08 | 16.78 |
| Alt2 | 104.25 | 109.20 | 104.68 | 100.65 | 96.96 | 14.84 | 17.44 | 17.47 | 18.21 | 21.15 |
| Alt3 | 99.60 | 105.18 | 101.70 | 97.91 | 92.66 | 16.24 | 16.87 | 17.54 | 18.41 | 25.84 |
| Alt4 | 100.95 | 92.50 | 85.19 | 81.78 | 86.30 | 16.96 | 18.23 | 20.19 | 21.85 | 20.16 |
| Alt5 | 101.62 | 93.92 | 90.37 | 86.41 | 86.16 | 15.60 | 19.00 | 18.85 | 22.29 | 23.52 |
| Alt6 | 99.13 | 93.81 | 86.37 | 84.25 | 83.89 | 16.24 | 16.85 | 18.55 | 19.01 | 21.79 |
| Alt7 | 99.80 | 96.28 | 85.44 | 78.63 | 81.14 | 14.93 | 16.79 | 19.44 | 22.26 | 21.19 |
| Alt8 | 102.43 | 95.08 | 85.06 | 77.38 | 75.51 | 16.37 | 18.17 | 20.99 | 25.19 | 25.12 |
| Alt9 | 101.09 | 93.37 | 85.89 | 82.85 | 84.06 | 16.24 | 14.83 | 18.17 | 21.22 | 21.76 |
| Alt10 | 100.64 | 92.63 | 82.76 | 81.28 | 79.85 | 16.01 | 17.48 | 17.99 | 19.03 | 21.52 |

TABLE 4 Results of *t*-Tests and ANOVA Tests (*p*-values)

| Section (m) | Alternatives | Alt1 | Alt2 | Alt3 | Alt4 | Alt5 | Alt6 | Alt7 | Alt8 | Alt9 | Alt10 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| −500 | Alt1 | 1 | 0.118 | 0.428 | 0.816 | 0.913 | 0.194 | 0.414 | 0.611 | 0.873 | 0.669 |
| | Alt2 | | 1 | **0.003** | **0.009** | **0.028** | **<0.001** | **0.006** | 0.253 | **0.016** | **0.013** |
| | Alt3 | | | 1 | 0.273 | 0.252 | 0.738 | 0.882 | 0.161 | 0.353 | 0.523 |
| | Alt4 | | | | 1 | 0.595 | 0.073 | 0.361 | 0.361 | 0.915 | 0.843 |
| | Alt5 | | | | | 1 | 0.103 | 0.251 | 0.595 | 0.722 | 0.550 |
| | Alt6 | | | | | | 1 | 0.561 | **0.049** | 0.140 | 0.431 |
| | Alt7 | | | | | | | 1 | 0.073 | 0.455 | 0.685 |
| | Alt8 | | | | | | | | 1 | 0.500 | 0.401 |
| | Alt9 | | | | | | | | | 1 | 0.797 |
| | Alt10 | | | | | | | | | | 1 |
| | ANOVA test | | | | | | | | | | |
| | *F*-statistic = 0.201 | | | | | | | | | | |
| | *p*-value .994 | | | | | | | | | | |
| −250 | Alt1 | 1 | 0.720 | 0.144 | **<0.001** | **<0.001** | **<0.001** | **<0.001** | **<0.001** | **<0.001** | **<0.001** |
| | Alt2 | | 1 | **0.041** | **<0.001** | **<0.001** | **<0.001** | **<0.001** | **<0.001** | **<0.001** | **<0.001** |
| | Alt3 | | | 1 | **<0.001** | **0.003** | **<0.001** | **<0.001** | **<0.001** | **<0.001** | **<0.001** |
| | Alt4 | | | | 1 | 0.555 | 0.323 | 0.057 | 0.132 | 0.640 | 0.939 |
| | Alt5 | | | | | 1 | 0.959 | 0.318 | 0.626 | 0.810 | 0.637 |
| | Alt6 | | | | | | 1 | 0.222 | 0.560 | 0.808 | 0.599 |
| | Alt7 | | | | | | | 1 | 0.402 | 0.156 | 0.142 |
| | Alt8 | | | | | | | | 1 | 0.425 | 0.233 |
| | Alt9 | | | | | | | | | 1 | 0.713 |
| | Alt10 | | | | | | | | | | 1 |
| | ANOVA test | | | | | | | | | | |
| | *F*-statistic = 3.541 | | | | | | | | | | |
| | *p*-value <.001 | | | | | | | | | | |
| −150 | Alt1 | 1 | **0.030** | **0.003** | **<0.001** | **<0.001** | **<0.001** | **<0.001** | **<0.001** | **<0.001** | **<0.001** |
| | Alt2 | | 1 | 0.251 | **<0.001** | **0.001** | **<0.001** | **<0.001** | **<0.001** | **<0.001** | **<0.001** |
| | Alt3 | | | 1 | **<0.001** | **0.002** | **<0.001** | **<0.001** | **<0.001** | **<0.001** | **<0.001** |
| | Alt4 | | | | 1 | 0.056 | 0.418 | 0.885 | 0.946 | 0.714 | 0.271 |
| | Alt5 | | | | | 1 | 0.120 | **0.030** | 0.088 | 0.084 | **0.005** |
| | Alt6 | | | | | | 1 | 0.640 | 0.499 | 0.737 | 0.066 |
| | Alt7 | | | | | | | 1 | 0.806 | 0.839 | 0.275 |
| | Alt8 | | | | | | | | 1 | 0.694 | 0.353 |
| | Alt9 | | | | | | | | | 1 | 0.118 |
| | Alt10 | | | | | | | | | | 1 |
| | ANOVA test | | | | | | | | | | |
| | *F*-statistic = 6.651 | | | | | | | | | | |
| | *p*-value <.001 | | | | | | | | | | |
| −75 | Alt1 | 1 | **0.002** | **<0.001** | **<0.001** | **<0.001** | **<0.001** | **<0.001** | **<0.001** | **<0.001** | **<0.001** |
| | Alt2 | | 1 | 0.324 | **<0.001** | **0.003** | **<0.001** | **<0.001** | **<0.001** | **<0.001** | **<0.001** |
| | Alt3 | | | 1 | **<0.001** | **0.002** | **<0.001** | **<0.001** | **<0.001** | **<0.001** | **<0.001** |
| | Alt4 | | | | 1 | 0.144 | 0.218 | 0.114 | **0.035** | 0.622 | 0.843 |
| | Alt5 | | | | | 1 | 0.471 | **0.028** | **0.016** | 0.221 | 0.063 |
| | Alt6 | | | | | | 1 | **0.030** | **0.012** | 0.433 | 0.090 |
| | Alt7 | | | | | | | 1 | 0.540 | 0.076 | 0.365 |
| | Alt8 | | | | | | | | 1 | **0.022** | 0.201 |
| | Alt9 | | | | | | | | | 1 | 0.396 |
| | Alt10 | | | | | | | | | | 1 |
| | ANOVA test | | | | | | | | | | |
| | *F*-statistic = 7.057 | | | | | | | | | | |
| | *p*-value < .001 | | | | | | | | | | |
| 0 | Alt1 | 1 | **0.001** | **<0.001** | **<0.001** | **<0.001** | **<0.001** | **<0.001** | **<0.001** | **<0.001** | **<0.001** |
| | Alt2 | | 1 | 0.150 | **<0.001** | **0.005** | **<0.001** | **<0.001** | **<0.001** | **<0.001** | **<0.001** |
| | Alt3 | | | 1 | **0.026** | 0.083 | **0.006** | **<0.001** | **<0.001** | **0.006** | **0.001** |
| | Alt4 | | | | 1 | 0.958 | 0.203 | **0.006** | **<0.001** | 0.233 | **0.010** |
| | Alt5 | | | | | 1 | 0.395 | 0.077 | **0.004** | 0.475 | **0.045** |
| | Alt6 | | | | | | 1 | 0.129 | **0.002** | 0.913 | **0.031** |
| | Alt7 | | | | | | | 1 | 0.054 | 0.149 | 0.556 |
| | Alt8 | | | | | | | | 1 | **0.001** | 0.118 |
| | Alt9 | | | | | | | | | 1 | **0.012** |
| | Alt10 | | | | | | | | | | 1 |
| | ANOVA test | | | | | | | | | | |
| | *F*-statistic = 5.372 | | | | | | | | | | |
| | *p*-value < .001 | | | | | | | | | | |

NOTE: Boldface indicates statistically significant values with 5% level of significance, underline indicates statistically significant values with 10% level of significance.

versus Alt3) did not significantly affect speed behavior. In the alternatives with perceptual treatments (Alt4-10) mean speeds were lower than in the base tangent and in the intersections without treatments ($p < .001$). Speed reductions ranged between 20 km/h and 28 km/h in relation to the base tangent (Alt1), and between 14 and 22 km/h in relation to the base intersection (Alt2). The raised median island produced the greatest speed reduction.

In the section −75 m, results were similar to those of section −150 m. The colored intersection area (Alt8) produced the lower mean speed, which was significantly lower ($p < .05$) than in the following alternatives with perceptual treatments: optical speed bars (Alt4), transverse rumble strips (Alt5), peripheral speed bars (Alt6), and painted median island (Alt9).

In the center of the intersection, perceptual treatment alternatives showed significant mean speed reductions ranging between 26 km/h and 37 km/h in relation to the base tangent (Alt1) and between 11 km/h and 21 km/h in relation to the base intersection (Alt2). In the colored intersection area (Alt8) mean speed was significantly lower than in all the other alternatives, except the raised median island (Alt10).

### Discussion of Results

Results of the cluster analysis and the statistical tests consistently showed that the speed behavior in the tangents was significantly affected by the presence of the intersections and by the perceptual treatments (see Figure 6).

Intersection without perceptual treatments significantly affected drivers' speed in the 250 m preceding the intersection. The two intersections with different sight distance did not exhibit different behavior. As far as this result is concerned, it must be considered that both intersections had sight distance greater than the average sight distance in the existing roads. In the center of the intersection, the mean speed reduction was equal to 16 km/h for the base intersection.

Perceptual treatments helped the drivers to detect the intersection earlier and to slow down, thus providing the drivers with better readability of the intersection and more time to perform evasive maneuvers. In the section −250 m, perceptual treatments were highly effective in reducing speeds ($p < .001$), and the differences compared with the base intersection ranged between 13 km/h and 17 km/h. Maximum speed reduction was observed 75 m before the intersection and was equal to 23 km/h (23%).

Even if all the alternatives were highly effective, some differences between the alternatives deserve attention. Dragon teeth markings (Alt7), colored intersection area (Alt8), and raised median island (Alt10) performed better than the other perceptual treatments. Optical speed bars (Alt4), rumble strips (Alt5), peripheral speed bars (Alt6), and painted median island (Alt9) showed similar effectiveness. The raised median island performed slightly better far from the intersection because it was perceived before the other alternatives. The colored intersection area (Alt8) performed the best close to the intersection (separate cluster in the last 75 m) because it was perceived very well when drivers were near the intersection.

## CONCLUSIONS

The aim of the paper was to investigate, by means of a dynamic driving simulator experiment, road users' behavior at rural intersections in relation to different perceptual measures designed for increasing hazard detection. In the experiment 10 different tangent configurations were tested. The VERA high-fidelity dynamic-driving simulator, operating at the TEST Road Safety Laboratory located in Naples, Italy, was used. Results were analyzed using two different

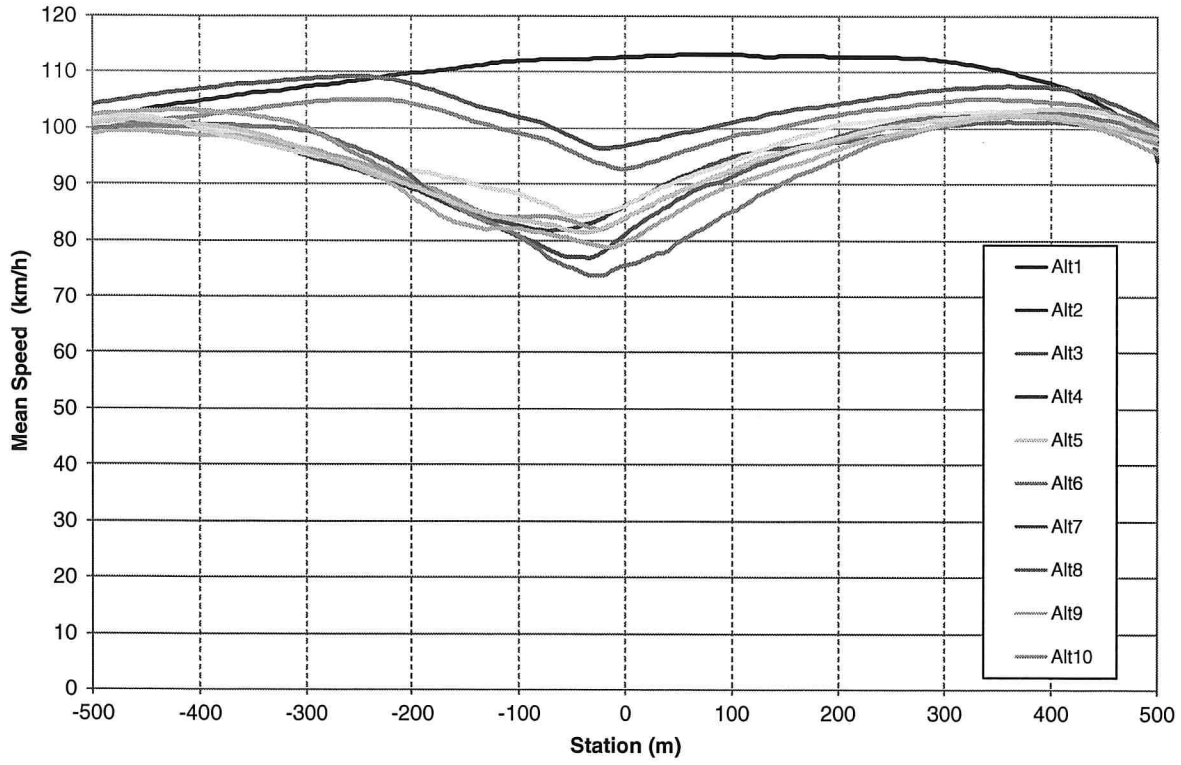


FIGURE 6 Speed profile.

approaches: (*a*) explorative description of data by cluster analysis and (*b*) inferential procedures about population using statistical tests.

Results of the cluster analysis and the statistical tests consistently showed that the speed behavior in the tangents was significantly affected by the presence of the intersections and the perceptual treatments. Intersection without perceptual treatments significantly affected driver speeds in the 250 m preceding the intersection. Perceptual treatments helped the driver to detect the intersection earlier and to slow down. Dragon teeth markings, colored intersection area, and raised median island performed better than the other perceptual treatments. They produced significant average speed reduction in the 150 m preceding the intersection, ranging between 16 km/h and 23 km/h. The raised median island performed slightly better far from the intersection, whereas the colored intersection area performed the best close to the intersection.

Because perceptual measures were effective in reducing speed and in detecting the intersection earlier, it can be argued that they improve the drivers' adaptation to the road environment and have the potential to reduce crashes. Study results support real-world implementation of perceptual measures in rural intersections because they are low-cost, fast implementation measures with a high potential to be cost-effective.

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*The Traffic Control Devices Committee peer-reviewed this paper.*

# Evaluating Innovative Ideas in Pedestrian Signing for Temporary Traffic Control

Brooke R. Ullman and Gerald L. Ullman

**A laboratory study of human factors was conducted to evaluate enhanced signing options that provide pedestrians in temporary traffic control areas with improved information on sidewalk closures. Researchers believed that improving the pedestrian information provided at work zones could result in better public compliance with signing. However, there were concerns that some pedestrian signing alternatives might be interpreted incorrectly by drivers as pertaining to their travel. As a result of these concerns, researchers conducted a human factors study from the perspectives of a pedestrian and a driver. The following are several key issues identified during this research: (*a*) none of the pedestrian signing alternatives affected driver understanding that the signs were intended only for pedestrians; (*b*) from the pedestrian perspective, the use of an orange background with black text for signs resulted in a higher level of participants indicating that they would react to the sign information (i.e., cross the street to a different sidewalk) without adversely affecting their comprehension of the sign meaning; (*c*) the action phrase "Use Other Side" was more intuitively understood by pedestrians than "Cross Here"; and (*d*) the inclusion of a distance reference to the closure is recommended for advance warning signs for pedestrians.**

According to statistics from the U.S. Department of Transportation, approximately 14% of work zone fatalities are pedestrians (*1*). The proper handling of pedestrian movements in temporary traffic control is one means of improving the safety of pedestrians in work zones. The design of the pedestrian component of temporary traffic control (TTC) needs to be a significant consideration in the overall TTC design, particularly when the work zone is located in an urban or suburban area. However, the task of accommodating pedestrians in temporary traffic control situations is challenging. Conditions in these areas are constantly changing, and no single set of traffic control devices can satisfy all conditions. Many variables, such as type of work, location of work, road type, geometrics, traffic volumes, and pedestrian demand, affect the pedestrian information needs at temporary traffic control areas.

The *Manual on Uniform Traffic Control Devices* (MUTCD) provides a few typical applications for pedestrian temporary traffic control treatments; however, these are applicable mainly at urban intersections (*2*). In addition, the information provided to pedestrians in these situations is limited to sidewalk closed information. Researchers hypothesized that pedestrian informational needs reach beyond this basic information, but could still be met through enhanced signing options. It was also believed that an improvement in pedestrian information provided at work zones could prompt better pedestrian compliance with signing. However, researchers did have concerns that drivers might view an enhanced pedestrian sign and incorrectly assume it was intended for motorists. Therefore, to gain perspective on these issues a human factors laboratory study was conducted using pedestrian and motorist points of view.

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## OBJECTIVE

The primary objective of the human factors laboratory study was to gauge public comprehension of current and innovative signs for use in guiding pedestrians in and around work zone areas.

## STUDY DESIGN

Researchers conducted a human factors study to evaluate alternatives for sidewalk closed signs in and around work zone areas. Results of previous human factors studies, along with input from state departments of transportation and major cities in the United States, were used as a basis for the experimental design of this study (*3*). From this, researchers decided to evaluate several specific features to improve the information provided to pedestrians. These features included

- Shape (rectangle versus diamond),
- Color (white versus orange),
- Symbol use (text directions versus symbol, arrows versus no arrows),
- Action phrasing ("Cross Here" versus "Use Other Side," other required alternate route wording), and
- Distance wording (measured distance versus landmarks).

### Study Protocol

The study was conducted as a laptop-based survey with open-ended questions. Researchers evaluated the sign options from two different perspectives: a driver's and a pedestrian's. Participants were told to assume they were either a pedestrian or a driver when they viewed the signs in the survey. Sign images were embedded into a real-world picture from either the pedestrian or the driver point of view as appropriate for the instructions the participant received. This was done to give participants a more realistic visual approach to the signs. In addition, to try to avoid bias in information interpretation,

researchers were careful not to indicate the target audience for the signs (i.e., pedestrians).

The pedestrian perspective survey included comprehension, comparison, and phrase preference sections. Each participant saw three of the sign alternatives in the comprehension portion of the survey. Participants did not evaluate all sign options because of time constraints (i.e., keeping survey duration under 15 min) and to avoid learning effects. The following questions were used in the comprehension portion of the survey:

1. What information is this sign providing to you?
2. Based on the information provided on the sign, what action would you take (if any)? Why?
3. Would you change anything about the sign to improve it? If yes, what?

Participant evaluations also included five sign comparisons and three phrase preferences. In the comparison and preference sections, to determine participants' reactions to the sign alternatives, researchers used simple yes/no or option selection questions followed by a request to explain the response. A total of four different survey instruments were used for the pedestrian perspective.

The driver perspective study was based primarily on interpretation (or comprehension) of the signing alternatives. A total of five signs were included in the evaluation, with each participant viewing three of these signs. The following questions were used during this evaluation:

1. What does this sign mean to you as a driver?
2. Does this information affect you as a driver? Y/N
   - If yes, how?
   - If no, who do you think it affects? How are they affected?

Two different survey instruments were used for the driver perspective.

### Participants

The research team recruited a total of 668 participants from six different locations in Texas for this study. The only criterion for the participants recruited was that they be over the age of 18. Researchers attempted to recruit a sample of pedestrians on the basis of a demographic sample of the population of Texas with regard to gender, age, and education level. Overall, the sample recruited approximated the Texas driver demographics reasonably well.

### Sign Alternatives

The eight sign alternatives used in this study are shown in Figure 1; only five of these were used for the driver's perspective survey (i.e., those with a driver survey sample number listed underneath them). Also included in this figure is the number of participants who viewed each sign for the pedestrian and driver perspective studies. The numbers vary as a result of the randomization of the signs assigned to each version of the survey. Some signs were viewed more often because there were fewer alternatives being examined in their design category (such as symbol signs). The intended meanings of the signs are all the same: the sidewalk ahead is closed and pedestrians are to cross the street at the location of the sign.

### Data Analysis

The initial phase of the analysis was to determine participant comprehension of the signs. Researchers used a standard understanding level of 85% as the baseline criterion for a sign being within an acceptable comprehension level for use in the field. In addition, researchers performed confidence interval tests ($\alpha = 0.05$) and a Bernoulli test of proportions to determine whether the identified differences, from 85% and between two comprehension levels, were statistically significant.

## RESULTS

### Pedestrian Perspective

#### *Sign Interpretation*

The purpose of the initial evaluation in this study was to determine how pedestrians interpreted the information provided in the sign alternatives. This study was conducted to ensure that pedestrians could quickly interpret the given sign and use the information to effectively travel to their destination. Table 1 shows the results of this analysis.

Signs 1, 2, and 3 ("Use Other Side" with arrow, "Use Other Side" without arrow, and "Cross Here" with arrow) had the best results, with 94% of participants correctly interpreting each of these signs. Signs 6 and 6a (symbol signs) had the lowest comprehension levels at 80% and 74%, respectively. Researchers believe that the lack of wording concerning what action to take was confusing to many participants. In addition, the arrow included on these signs without routing directions influenced participants to believe that the opposite sidewalk may be closed and caused them to doubt what route they should take. This would imply that the inclusion of an action phrase with an arrow (such as in Sign 3) is critical to ensuring that pedestrians can use the provided information.

All signs except 6 and 6a have a comprehension percentage that is greater than 85%. In addition, Sign 6 is not statistically different from this comprehension level on the basis of a confidence interval test. A test of proportions was also used to determine whether the lower percentage signs were statistically different from Signs 1, 2, and 3 (which were the best understood). From this test, researchers determined that Signs 6 and 6a are significantly different from the best comprehension level. On the basis of this analysis it is not recommended that either of these signs be used in the field.

#### *Action*

The second component of an effective sign is that it prompts pedestrians to take the desired action (e.g., cross the street) on the basis of the information they gain from the sign. For the participants who correctly interpreted the signs, Table 2 shows the actions they indicated they would take.

From this information, it can be seen that once people understand the sign, the percentage of pedestrians who will follow the direction (or implied action) to cross the street is very high. However, for two of the signs (Signs 4 and 6), the levels of participants indicating they would follow the desired action were much lower than for the others. These signs have some fairly obvious flaws that would make them less understood than others with regard to what action a pedestrian should take. With Sign 4, there is no directional arrow provided

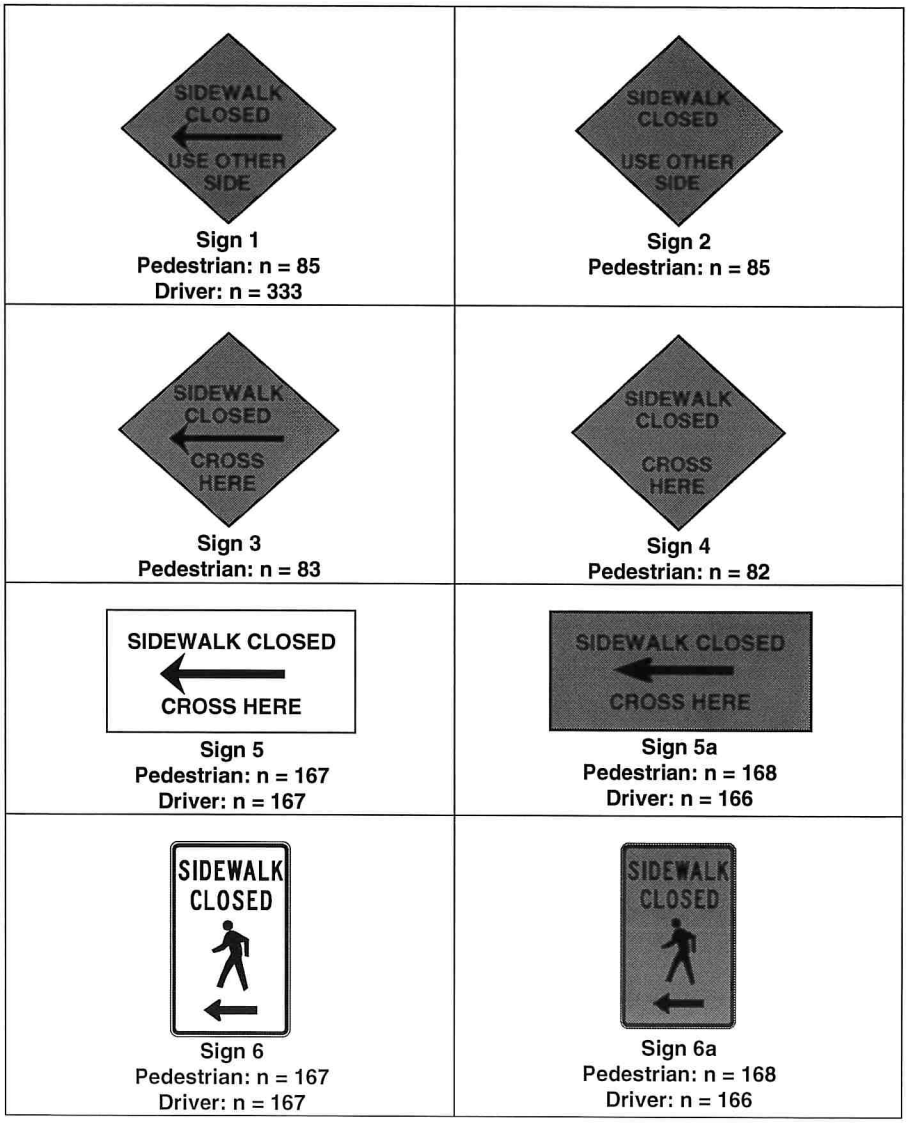



FIGURE 1 Pedestrian signing alternatives.

with the instruction "Cross Here" to assist people in understanding where they should cross. Consequently, a greater number of the participants, 17%, indicated that they would continue ahead on their original sidewalk.

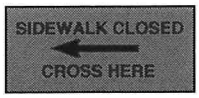
For Signs 6 and 6a, it was noted in the previous discussion that these signs had low levels of understanding. However 6a was found to have an acceptably high percentage of people indicating they would follow the actions prescribed by the sign when the sign was understood. Therefore, researchers believe that the lower level of desired action stated for Sign 6 is attributable to the color change for this design. From results that will be discussed later in this paper, researchers believe that some of the increase in desired reaction by pedestrians with Sign 6a over Sign 6 is due to the orange color. Participants indicated in later questions that orange was a more attention getting sign and a color that must be complied with.

All other signs had very high percentages of participants stating that they would cross the street. This implies that if the comprehension levels are sufficient, agencies should expect to see very good acceptance of the given (or implied) instructions. For this analysis, it was found that Sign 3 had the highest percentage of participants indicating that they would cross the street; however, on the basis of a test of proportions this number was not statistically different from any of the others except Signs 1, 4, and 6. This would indicate that if pedestrian reaction was being used as a deciding factor for the selection of signs, any of Signs 2, 3, 5, 5a, or 6a would be acceptable. However, recommendations from this report will be based on multiple deciding factors (not least of all the comprehension levels discussed previously).

### *"Use Other Side" Versus "Cross Here"*

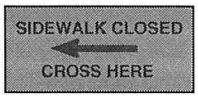
There are several specific design variations in these signs that researchers were evaluating to determine which would be better to use. First, the wording of the action term was evaluated. More specifically, this study looked at whether it would be better to use "Cross Here" or "Use Other Side" in pedestrian signing. Researchers found that when an arrow was included on the sign, there was very little

TABLE 1 Pedestrian Interpretation

| Sign Option | Interpretation Percentages: Sidewalk Closed and Cross Road | Opposite Sidewalk Closed | Crosswalk Closed | Sidewalk Closed—Don't Know What Route to Take | Do Not Know | Other |
|---|---|---|---|---|---|---|
| Sign 1 | 94 | | 2 | | 2 | 2 |
| Sign 2 | 94 | | 2 | 2 | 2 | |
| Sign 3 | 94 | 1 | 1 | 4 | | |
| Sign 4 | 89 | 1 | 1 | 6 | 1 | 2 |
| Sign 5 | 87 | 5 | 5 | 2 | | 1 |
| Sign 5a   | 89 | 4 | 4 | 2 | 1 | |
| Sign 6 | 80[a] | 9 | 2 | 7 | 2 | |
| Sign 6a | 74[b] | 8 | 4 | 11 | 1 | 2 |

[a]Is statistically different from highest comprehension level.
[b]Is statistically different from highest comprehension level and 85 percent.

TABLE 2 Pedestrian Actions

| Sign Option | Interpretation Percentages | | | | |
|---|---|---|---|---|---|
| | Cross Street | Continue Straight | Take a Different Alternate Route | Do Not Know | Other |
| Sign 1 | 85[a] | 11 | 4 | | |
| Sign 2 | 91 | 3 | 5 | | 1 |
| Sign 3 | 95 | 3 | 1 | | 1 |
| Sign 4 | 75[a] | 17 | | 7 | 1 |
| Sign 5 | 88 | 8 | 4 | | |
| Sign 5a   | 93 | 2 | 4 | | 1 |
| Sign 6 | 81[a] | 9 | 8 | 2 | |
| Sign 6a | 91 | 4 | 3 | | 2 |

[a]Is statistically different from highest level.

interpretation difference between the two phrases. The "Cross Here" sign had a higher number of participants who stated they would follow the given action. However, closer analysis of the responses for the "Use Other Side" sign showed that the greatest number of people indicating that they would not cross the street were continuing straight because they did not see a problem ahead or they did not feel that the absence of a sidewalk would influence their actions (they could walk in the grass), not because they did not understand the given action. Given this interpretation, neither of these phrases is significantly better understood than the other when an arrow is included.

However, when an arrow was not included on the sign, "Use Other Side" was much better understood and had a higher level of

people indicating that they would react (i.e., cross the street) than with "Cross Here." Again, the actions stated for the "Cross Here" sign were primarily to continue ahead; however, in this case it was not as clear that people were just ignoring the sign, but that they were unsure of their route choices.

"Cross Here" and "Use Other Side" without an arrow were also evaluated in a direct comparison of preference. For this analysis, it was found that participants were split with regard to which phrase they preferred (51% and 49%, respectively). In both cases, participants who preferred the option thought that their selected form of the instruction was more descriptive or clear as to what action they should take. This comparison does not reveal a clear indication as to which phrase should be used on the basis of preference; therefore, comprehension was used for this decision. In that case, the findings imply that "Use Other Side" is more intuitively understood by pedestrians.

### Arrow Versus No Arrow

The general difficulty with the use of arrows for "sidewalk closed" messages is that pedestrians can interpret these to be directed at the sidewalk that is closed and not at the route they should take. In this case, the interpretations of signs with and signs without arrows were relatively similar; however, there were some differences with regard to the participants' described actions. In the case of the arrow and non-arrow versions of the sign that had "Use Other Side" as directions, there was not a statistically significant difference in the actions people would take. However, more people chose to disregard the sign with the arrow than without, implying that the arrow did not improve people's willingness to comply. Conversely, the signs that contained the phrase "Cross Here" as direction produced a significantly better reaction from participants (i.e., they followed the directions) when an arrow was included on the sign. Of the people who did not indicate they would cross the street with the non-arrow sign, the majority indicated that either they did not know what action they should take or that there was no direction given as to where they should cross. This analysis would imply that the need for an arrow is directly tied to the phrasing used on the sign. In cases in which "Cross Here" is the desirable phrasing, an arrow must be used; however, in the case of "Use Other Side" the arrow does not improve signing interpretation.

### Words Versus Symbols

An additional sign format evaluated included the use of a symbol (Signs 6 and 6a) versus traditional wording as a means of indicating what action the participant should take. Researchers found that the interpretations of Signs 6 and 6a as compared with the other signs evaluated were both very low, implying that the symbol was not as well understood as the traditional wording. As shown in Table 1, when the action direction was removed to include the symbol, pedestrians more often stated that the sign provided them with the information that the sidewalk was closed but did not indicate a different route to take. Also, compared with the traditional text signs, with the symbol signs participants were more confused about whether the arrow was pointing to the closed sidewalk or to the direction they should travel.

Researchers used Signs 5 and 6 (white versions of the traditional and symbol signs) to conduct a preference evaluation of the symbol alternative. In this comparison, 62% of participants indicated a preference for the symbol sign format. These people stated that they liked the symbol because it was recognizable. Unfortunately, this recognition of the sign did not help with their interpretation ability in this situation. Some of this could be due to the fact that although people indicated that they liked the pedestrian sign, they did not understand that it was intended to provide them with a direction or route to follow based on the given arrow. Therefore, participants appeared to select their preferences on the basis of an unintended meaning for the sign. On the basis of the full interpretation and preference information available for the symbol sign, researchers believe that if the symbol was incorporated into a different design in which the action required was more obvious (i.e., inclusion of action text), it may be a viable alternative.

### Color

Currently, pedestrian signs included in the MUTCD are primarily white with black text. However, on the basis of previous studies, researchers had reason to believe that a change to orange signs with black text might inspire a better understanding and reactions from pedestrians on the basis of the fact that orange is an identifiable color for work zones (*3*). Both orange and white signs were used in this study to evaluate whether participants interpreted the intended meaning of the signs differently on the basis of color. In addition, researchers queried participants to determine whether they thought one color was more likely to cause them to comply with the suggested action. For this analysis, participants interpreted two sign formats: one white sign and one orange sign with the symbol format and one white sign and one orange sign with the traditional wording format (Signs 5/5a and 6/6a).

For these given sign pairs, researchers did not find a significant difference in the interpretation or comprehension of the signs. However, when the participants' perceived actions are evaluated, there is a decided increase for the number of participants who indicate they will follow the given instructions for both orange signs over their white alternatives. In both cases, participants were more likely to comply with the intended action (cross the street) for the orange version of the sign versus the white. This finding is significant particularly when looking at some of the direct comparison information for this sign feature. Researchers asked participants which color would imply that they were breaking the law if they disobeyed the instruction given. Sixty-five percent of participants indicated that orange was the color that implied they were breaking the law as compared with only 13% for white. This is in direct opposition to the intended meanings of these colors in transportation application (*2*). Figure 2 shows the breakdown of responses for this question.

### Shape

The final sign feature being considered is shape; researchers directly evaluated what effect shape has on interpretation of or reaction to signs. This survey included rectangular- and diamond-shaped signs to evaluate whether there was a difference in how they were interpreted by pedestrians. Interestingly, the difference in comprehension for signs that had different shapes was not statistically significant. Similarly, although there was a small difference in the actions stated by participants, it was also not significant. This would imply that the shape of the sign did not overly influence participant interpretations. Supporting this conclusion is the information gained from the comparison of the different shapes. In the direct comparisons of shape, 90% of participants did not believe that there was a difference in the

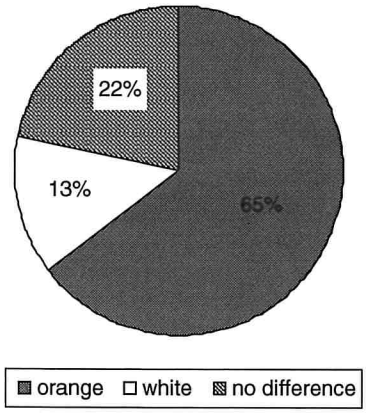



FIGURE 2 Interpretation of regulatory nature based on color.

meanings of the signs. Therefore, shape should not be a deciding factor in the design of pedestrian signage with regard to whether or not the information will be understood by the user.

### *Wording Alternatives*

Researchers included a final section in the pedestrian perspective survey that looked at different options related to alternate route and distance wording that could be included on temporary traffic control for pedestrians. Researchers wanted to identify how pedestrians perceive different wording that could be used in the field. In the alternative route evaluation, researchers looked at the following five different terms to see whether pedestrians believed they were required to use a different route to reach their destination:

- "Detour,"
- "Alternate Route,"
- "Alternate Path,"
- "Cross Here," and
- Use Other Side.

Participants selected as many of these options as they believed applied. The table below shows what percentage of participants selected each option:

| *Option* | *Percentage* |
|---|---|
| Detour | 87 |
| Alternate route | 64 |
| Use other side | 62 |
| Alternate path | 52 |
| Cross here | 43 |

For this evaluation, researchers found that "Detour" was the most common response, with 87% of participants selecting it to mean they were required to use a different route. This was followed by "Alternate Route" (64%) and "Use Other Side" (62%). However these were significantly lower than "Detour" in regard to the number of responses. The term "Cross Here" had the lowest percentage of interpretation. This indicated to researchers that the phrase "Cross Here" is believed largely to be a suggestion to pedestrians and not a directive.

In addition, researchers investigated forms of providing information to pedestrians in regard to how far ahead a temporary traffic control area is located. Figure 3 illustrates the sign that was shown to participants when they were asked to identify what information should be included in the blank where the "?" was located.

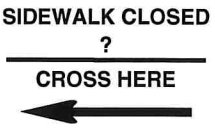



FIGURE 3 Distance question sign.

They were given four alternatives to select from for this task, two giving landmark points and two giving distances. The participant was told that all information alternatives represented the same walking distance for the pedestrian. Researchers included this information to try to reduce bias created by unfamiliarity of the landmarks versus the distance understanding. Table 3 shows the results for which option participants felt would be best to include on the sign.

As can be seen, the distance options were much better received than those giving landmarks. Researchers noted while administering the survey that most participants were assuming that they were unfamiliar with the area during this selection process. Therefore, the landmark category options were less well received. For the distance category options, the preference was greater for "1 Block Ahead"; however, this was not statistically different from "200 ft Ahead" on the basis of a test of proportions. For both cases, participants who selected each option indicated primarily that it was easier to determine how far it was. On the basis of these user preference findings, researchers would suggest the use of distance references over landmarks for pedestrian signing. In regard to the format of that distance, often that is determined more by the location than by preference. If there are regular blocks in the area, blocks would most likely produce the best results in regard to pedestrian comprehension; however, when this is not true, distances in feet should be effective in communicating with pedestrians.

### *Sign Improvements*

Following the interpretation of each sign, participants were asked what improvements they would suggest for the sign they had just viewed. The signs with the greatest number of suggestions offered were Signs 6 and 6a. In both cases, the most common improvement suggested by participants (17% for each sign) was to indicate what action should be taken.

Sign 4 had the highest percentage of participants who selected a single improvement. Twenty-seven percent of participants suggested the addition of an arrow, echoing strongly the findings of the interpretation and actions for this sign in instances in which participants were having difficulty interpreting "Cross Here" without further guidance (i.e., an arrow). Sign 2, which also provides an

TABLE 3 Percent Selecting Distance Alternatives

| Category | Option | Option (%) | Category Total (%) |
|---|---|---|---|
| Landmarks | After Green Street | 12 | 19 |
| | Past McDonalds | 7 | |
| Distances | 1 block ahead | 44 | 81 |
| | 200 ft ahead | 37 | |

action ("Use Other Side") without an arrow direction, also had a suggested addition; however, it was at a much lower rate of 13%.

### Driver Perspective

As mentioned previously, researchers were concerned that changes to shape and color of the pedestrian signing being evaluated could possibly cause motorist confusion in differentiating between signs that are intended for driving versus those intended for pedestrians. Also, researchers wanted to evaluate the wording "Use Other Side" to ensure that drivers could interpret this to mean pedestrians and not that the driver should change lanes. For that purpose, researchers developed a survey instrument to evaluate motorist interpretation of signing alternatives.

The signs included in the interpretation evaluation for drivers were Signs 1, 5, 5a, 6, and 6a. This allowed researchers to analyze driver reaction to traditional signing (Sign 5), orange signing (Signs 1, 5a, and 6a), diamond shape (Sign 1), "Use Other Side" (Sign 1), and pedestrian symbol (Signs 6 and 6a). For each of these signs, Table 4 shows the level of driver comprehension that the sign was intended for pedestrians.

As shown in the table, all signs evaluated had very high levels of comprehension and very little confusion that they were intended as anything other than pedestrian information. This indicates that none of the variables being examined (as outlined above) had an adverse impact on motorist understanding. This alleviates concerns that researchers had and implies that the design of pedestrian signs with relation to shape, color, text, or symbol can be based on the results of the pedestrian perspective survey.

## CONCLUSIONS AND RECOMMENDATIONS

Through the use of temporary traffic control planning, transportation agencies are able to provide a continuity of movement through an area that is affected by a work zone. This continuity is necessary for all public users including pedestrians. However, the MUTCD has very few typical applications that relate to pedestrian treatments in temporary traffic control situations. In addition, the information provided to pedestrians in these situations is limited. This research was conducted to evaluate different options that could be used in providing temporary traffic control information to pedestrians when a sidewalk is closed as a result of a work zone. Researchers did have concerns with respect to drivers misinterpreting the alternative designs for pedestrian signing; therefore this evaluation was conducted from a driver's and a pedestrian's point of view.

On the basis of information gained through previous human factors studies, researchers decided to conduct a human factors laboratory study to gain the public's input on different traffic control devices to meet pedestrian information needs. In this effort, researchers administered a survey to the general public to obtain interpretation and comprehension data for sign alternatives. In the alternative designs, the following elements were included for evaluation:

- Shape (rectangle versus diamond),
- Color (white versus orange),

TABLE 4 Driver Understanding

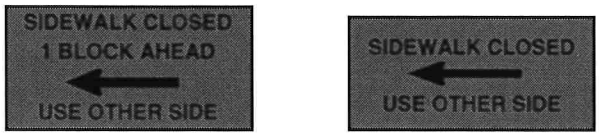
| Sign Option | Comprehension Percentages | | | | |
|---|---|---|---|---|---|
| | Intended for Pedestrians | Construction | Change Lanes | Stop | Don't Know |
| Sign 1 | 95 | 1 | 3 | | 1 |
| Sign 5 | 98 | | 1 | 1 | |
| Sign 5a | 96 | 1 | 1 | | 2 |
| Sign 6 | 100 | | | | |
| Sign 6a | 98 | | 1 | | 1 |

- Symbol use (text directions versus pedestrian symbol, arrows versus no arrows),
- Action phrasing ("Cross Here" versus "Use Other Side," required alternate route wording), and
- Distance wording (measured distance versus landmarks).

The key findings of this study are listed below:

- None of the evaluated changes affected driver understanding that the signs were intended only for pedestrians and did not require them to take specific driving actions.
- From the pedestrian perspective, the use of an orange background with black text for signs was found to produce an increase in pedestrian response in regard to following the directions or implied actions they had observed on the sign without adversely affecting their comprehension of the sign meaning.
- Shape had little to no impact on the interpretation of the sign; therefore, researchers are recommending staying with the standard rectangular layout for the "Sidewalk Closed" signs as currently included in the MUTCD.
- Action phrasing "Use Other Side" was found to be more intuitively understood by participants and is therefore recommended for use as the better option when directing pedestrians to an alternate route across the street.
- The inclusion of a distance reference on the sign was found to be preferable to participants in the survey over the use of nearby landmarks. Therefore, researchers would recommend including "1 Block Ahead" on advance warning signs when there are regular blocks in the area because this would most likely produce the best results in regard to comprehension of the distance. However, when blocks are not appropriate for the site, distances in feet should be used to express the distance to the closure area.

On the basis of this survey, researchers recommend the use of the signs shown in Figure 4 when there is a sidewalk closure due to a work zone. Both of these signs should have orange backgrounds with black characters (text).




(a) (b)

FIGURE 4 Recommended pedestrian signs: (*a*) advance warning sign and (*b*) at closure near crosswalk.

## ACKNOWLEDGMENTS

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*The contents of this paper reflect the views of the authors, who are responsible for the facts and the accuracy of the data. Contents do not necessarily reflect the official view or policies of FHWA or the Texas Department of Transportation.*

*The Traffic Control Devices Committee peer-reviewed this paper.*

# Assessing Benefits of Chevrons with Full Retroreflective Signposts on Rural Horizontal Curves

Jonathan M. Ré, H. Gene Hawkins, Jr., and Susan T. Chrysler

**Driving a horizontal curve requires a change in vehicle alignment and a potential reduction in speed. Curves may present a challenging situation during adverse conditions or to inattentive drivers. Chevron signs provide advance warning and positive guidance through the curve. Some agencies place supplemental retroreflective material on chevron signposts to enhance the signs' conspicuity and visibility. The objective of this study was to determine any incremental benefits in vehicle speed and lateral lane position when retroreflective material was applied to chevron signposts. This study evaluated three scenarios in a before-and-after experimental design. The before evaluation presented no vertical delineation (baseline). The two after evaluations consisted of a series of standard chevrons (chevrons) and chevrons with fully retroreflective posts (full-post chevrons). Traffic counters measured vehicle speed and lateral lane position at the point of curvature and midpoint of the curve at two separate study sites. Findings showed that chevrons and full-post chevrons moved vehicles away from oncoming traffic by about 15 in. Overall, there was little difference between the lateral position findings of the two chevron treatment scenarios. Chevrons achieved a 1.28-mph reduction in mean vehicle speed from the baseline evaluation, and full-post chevrons obtained a 2.20-mph reduction. The findings determined that the benefits of full-post chevrons were not substantial. Researchers recommend that the *Manual on Uniform Traffic Control Devices* (MUTCD) should continue to present full-post chevrons as an optional delineation tool. On the basis of this research, the researchers do not recommend any changes to the MUTCD.**

A horizontal curve requires a change in vehicle path alignment and a potential reduction in vehicle speed. The change from tangent alignment may present a challenging task during adverse driving conditions or to inattentive drivers. Delineation devices and horizontal curve treatments may aid and assist drivers in safe and efficient horizontal curve negotiation. Delineation treatments provide advance warning on the approach tangent and positive guidance throughout the curve. Chevron signs are a common type of delineation treatment and are placed on the outside of a curve. Some agencies have been placing supplemental retroreflective material on the chevron signposts to enhance the conspicuity and visibility of the sign.

The only standards that govern the application of retroreflective material on signposts are contained in the 2003 edition of the *Manual on Uniform Traffic Control Devices* (MUTCD). The MUTCD states in Section 2A.21 that "Where engineering judgment indicates a need to draw attention to the sign during nighttime conditions, a strip of retroreflective material may be used on regulatory and warning sign supports" (*1*). Agencies may use the additional retroreflective material as an optional treatment, and it is not required by the MUTCD. The option to use supplemental retroreflective sheeting on a signpost was first added in the 2000 edition of the MUTCD. The proposed rule for this change did not identify any research that supported its addition to the manual. Placing retroreflective material on chevron signposts (full-post chevrons) is relatively inexpensive, the installation is easy, and little maintenance cost is required. If the current option of applying retroreflective material to chevron signposts is to remain in the MUTCD, then it is a worthy endeavor to investigate the treatment in a more comprehensive study to ascertain whether there are additional benefits in both speed and lateral position.

The objective of this study was to compare the effects of chevrons and full-post chevrons with a baseline condition with no treatment, in a before and after experimental design, and to determine whether full-post chevrons achieved additional benefits over the baseline evaluation and the chevron evaluation. This objective was accomplished by analyzing three separate evaluation scenarios in a before and after experimental design at two separate curve sites. The three evaluation scenarios were used at each site to isolate the specific effects of the treatments. Evaluation scenarios consist of an existing baseline evaluation (before), an evaluation with standard chevrons (after), and an experimental evaluation with full-post chevrons (after). The analysis compared both chevron treatments with the baseline evaluation to identify any changes in vehicle operations. The order of chevrons and full-post chevrons was reversed at each site. Results from full-post chevrons were compared with results from chevrons to determine whether the added retroreflective material achieved additional benefits.

## BACKGROUND

Drivers must guide their vehicles safely through a horizontal curve. Vehicle guidance involves maintaining the proper speed and lane placement that does not conflict with roadway constraints or regulations. Vehicle guidance is one of the fundamental tasks in the Lunenfeld and Alexander positive guidance framework, which includes vehicle control, guidance, and navigation (*2*). Driver error in one of

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the three vehicle positive guidance tasks may increase the chance of a potential hazard. Error or a guidance breakdown on a horizontal curve is typically attributed to either improper speed selection or inadequate lateral lane position. Chevrons are placed on a horizontal curve to curtail improper speed and lateral position by providing advance warning and guidance.

In 1983 Niessner summarized several field studies that evaluated the effect of post-mounted delineators (PMDs) and chevrons on vehicle safety and crash rates (*3*). The summary analyzed results from eight different states. All reviewed studies were conducted in a before and after experimental design. Each study evaluated crash rates before and after the installation of delineation treatments. Niessner extracted from the studies that chevrons significantly reduced the fatal crash rate and PMDs significantly lowered run-off-the road crashes. The study concluded that both chevrons and PMDs were adequate devices for delineating horizontal rural curves.

The reduced crash rates may be attributed to improved vehicle operations that were shown in an Australian study conducted in 1983. Johnston evaluated the benefits of chevrons and PMDs in a closed-course test track (*4*). Delineation treatments were assessed by measuring vehicle lateral position, encroachment rates, and speed. Johnston showed that the curves without delineation treatments exhibited the least desirable vehicle operations. Curves using chevron signs achieved significantly lower vehicle speed during nighttime and superior lateral position results compared with curves with PMDs.

A study in Virginia conducted an open-road field evaluation that was similar to Johnston's study. Jennings and Demetsky compared the effects of chevrons, PMDs, and a road-edge delineator on horizontal curves (*5*). Each treatment was placed individually on five curves, and vehicle speed and lateral position data were collected at the point of curvature (PC) and the curve midpoint (MP). Results determined that all treatments shifted drivers away from the edge line on an outside curve. Jennings and Demetsky concluded that chevrons promoted a more centralized vehicle path, reduced encroachment rates, and lowered the lateral position variance.

The previous studies dealt with conventional or standard delineation devices. A study by Pietrucha et al. in 1996 investigated older drivers' curve perception when standard and enhanced delineation treatments were implemented (*6*). The researchers evaluated PMDs with a fully retroreflective viewing surface and *T*-post PMDs, which used a thin strip of retroreflective material from the top to the bottom. Both enhanced treatments achieved long detection distances by subjects. Pietrucha et al. reasoned that effective treatments exhibited large retroreflective targets and retroreflective material that extended from the top of the device to the ground.

Enhanced full-post chevrons were first assessed in a study conducted in 2003. Gates et al. evaluated a 4-in.-wide strip of fluorescent yellow prismatic sheeting that extended the entire length of a chevron signpost (*7*). The treatment was placed on all chevron signs at one rural horizontal curve. Vehicle speed was measured at two upstream tangent points, the PC and the MP. Overall, full-post chevrons achieved a slight speed decrease of 1.7 mph during twilight and 1.6 mph during the night. Researchers concluded that the "use of fluorescent yellow microprismatic materials on chevron posts or other curve delineation is recommended on an as-needed basis at spot locations where additional delineation is desired" (*7*).

A closed-course study of nighttime visibility of fully retroreflective post-mounted delineators and chevrons was conducted in coordination with the current project (*8*). The study showed that reflectorized posts had a slight benefit for chevrons and a much larger improvement for post-mounted delineators. Researchers believe that the post-mounted delineators have so little target value to begin with, reflectorizing the posts can really increase visibility; whereas standard chevrons have good visibility because of their size, so reflectorizing the posts provides only marginal gains.

Placing retroreflective material on signposts and enhanced PMD treatments has shown promise in past studies. The Pietrucha et al. (*6*) study assessed curve detection on a closed-course test track, and the Gates et al. (*7*) study measured vehicle speed only on a single roadway curve. This background review determined that there was a need to assess the effects of full-post chevrons on vehicle speed and lateral position in an open-road study with more than one test curve.

## STUDY DESIGN

Besides identifying a need to further investigate the full-post chevrons, the background review was an instrumental component in developing the study design for this project. Previous research helped to determine appropriate measures and techniques for the experimental approach, site selection, and delineation treatment application.

### Experimental Approach

The experimental approach measured vehicle operations in a before and after design, which identified changes in vehicle speed and lateral position that could be attributed to the chevron treatments. The study consisted of three separate evaluation scenarios: a before, a first after, and a second after. The before scenario was an existing baseline evaluation with no vertical delineation treatment. There were two treatment scenarios that consisted of standard chevrons (after) and experimental full-post chevrons (after). Researchers collected vehicle speed and lateral position data at each test site before the addition of a study treatment in the baseline evaluation. Following the baseline evaluation, chevron treatments were installed for the first after evaluation and vehicle data were collected at the same site in an identical manner. Afterward, the full-post chevron treatment was removed from Site 1 and installed on Site 2 for the second after evaluation.

Measures of effectiveness (MOEs) define the vehicle operations for each evaluation scenario. A comparison between the MOEs of two different scenarios reveals the changes in vehicle operation. The general testing hypothesis states that if there is a relationship between full-post chevrons and a beneficial change in MOEs, then it is possible to associate the treatment with traffic safety. MOEs included longitudinal components (speed) and lateral components (lateral lane position). The MOEs in this study were

- Mean lateral position,
- Lateral position standard deviation,
- Lane line encroachment rates,
- Mean speed,
- Speed standard deviation, and
- High vehicle speed.

### Site Selection

This study benefitted from resources of material, manpower, and time to assess the chevron treatments on two rural horizontal curves. It was highly important that both selected test sites be ideal and satisfactory. The first step in the selection process was to generate a preliminary

list of potential horizontal curve sites. Researchers established potential sites with basic criteria such as roadway classification, posted speed limits, and traffic volumes. The preliminary list included 170 potential curves near Bryan, Texas. Research personnel visited each potential site and digitally filmed the curve for later use. Geometric characteristics, traffic control devices, roadway features, and other relevant information were recorded in a spreadsheet for each curve.

After compiling the curve data, the researchers then generated a list of site selection criteria to systematically eliminate any curves that were not ideal. Site selection criteria took the following into account:

- Roadway geometry and design characteristics,
- Curve warning signs and advisory speed plaques,
- Interference from driveways or roadway objects in the immediate area, and
- The ability to safely install and maintain data collection equipment.

Two suitable sites were selected from the list of potential curves. Both were on rural sections of two-lane road; Site 1 was in a wooded area and Site 2 was surrounded by agricultural fields with no trees. Site 1 exhibited a deflection angle of 37.5° and a radius of 1,071 ft. Site 2 had a deflection angle of 45° and a radius of 1,238 ft. The travel lanes at both sites are about 10 ft in width; dimensions may vary a few inches depending on the pavement markings. There were paved shoulders along both roadways, and the width of the shoulder varied from 1 to 4 ft. Both sites are posted for a 70-mph speed limit during the day and a 65-mph speed limit during the night. Site 1 and Site 2 had advisory speed plaques of 45 and 50 mph, respectively. A comparison between the current advisory speeds and an alternate advisory speed method determined that Site 1 should have an advisory speed of 55 mph and that the advisory speed at Site 2 was appropriate. The alternate advisory speed was determined using a method in a 2007 study that took into account the 85th percentile tangent speed, curve radius, and superelevation (*9*).

### Delineation Treatment Application

All treatments in this evaluation were in accordance and complied with Texas Department of Transportation (DOT) and MUTCD standards. The retroreflective material for full-post chevrons was flexible prismatic fluorescent yellow sheeting, which matched the material used on the chevron sign faces. The sheeting was applied to a section of PVC pipe that was 2.5 in. in diameter and 4 ft in length. The retroreflective PVC pipe was placed over the 2¾-in. signpost and completely encircled the signpost. The retroreflective material was not applied directly to the signpost because removing the sheeting would leave adhesive residue that could collect dirt and debris. The retroreflective PVC pipe proved to be economical and very efficient for changing between chevrons and full-post chevrons.

The spacing of chevron treatments was based on the roadway delineation section of the Texas MUTCD. The Texas MUTCD details that chevron signs are placed throughout the curve between the PC and PT and that one chevron sign is placed on the entrance and exit tangent. Chevron signs were spaced 160 ft apart at both Site 1 and Site 2. Seven chevron signs were installed at Site 1, and nine chevron signs were installed at Site 2. Signs were located at a 12-ft offset from the nearest travel lane to the nearest edge of the sign. Figure 1 displays an image of full-post chevrons implemented at Site 1.

FIGURE 1 Full-post chevrons at Site 1.

## DATA COLLECTION AND ANALYSIS

This section documents the methods used to collect and analyze the vehicle speed and lateral position data. The preponderance of time and effort spent on this project was concentrated on these tasks.

### Data Collection Procedure

Vehicle speed and lateral position data were measured using a traffic classifier and three roadway sensors. Piezoelectric roadway sensors were placed on the roadway and detected the presence of a passing vehicle from the pressure of the tires. Piezoelectric sensors consist of a thin metallic wire, which was inserted into pocket tape that adhered to the roadway surface. Three piezoelectric sensors positioned in a pattern that resembles the letter "Z" collected the data and relayed it to the traffic classifier. Traffic classifiers store the vehicle data with an exact time stamp. Speed was calculated from the known distance between sensors and the time it took a vehicle to travel across that distance. Lateral position data were a calculated product from the known geometric proportions of a right triangle, vehicle speed, and time stamps. Data collection equipment was placed at the PC and MP curve locations. Researchers selected these curve locations because they are easily referenced and they provide uniform locations at all sites. Researchers measured data in the inside (right-handed) and outside (left-handed) curve directions. Inside and outside terminology was used in this study, as opposed to right- and left-handed, because right and left are relative terms depending on travel direction but inside and outside are ubiquitous. Figure 2 depicts a diagram of data collection layout and locations.

Control speeds were also measured approximately 1 mi upstream from the test sites on both approaches. The control speed was outside the influence of the treatments and was meant to indicate whether vehicle speed changed considerably between evaluation scenarios. Researchers determined that there were no substantial changes in control speeds between evaluation scenarios.

Following specific steps in the data collection process helped to ensure reliable and verifiable data. The steps in the data collection schedule were as follows:

- Collect baseline data for the before evaluation at both sites,
- Install chevron signs at both test curves and place full-post chevrons on Site 1,

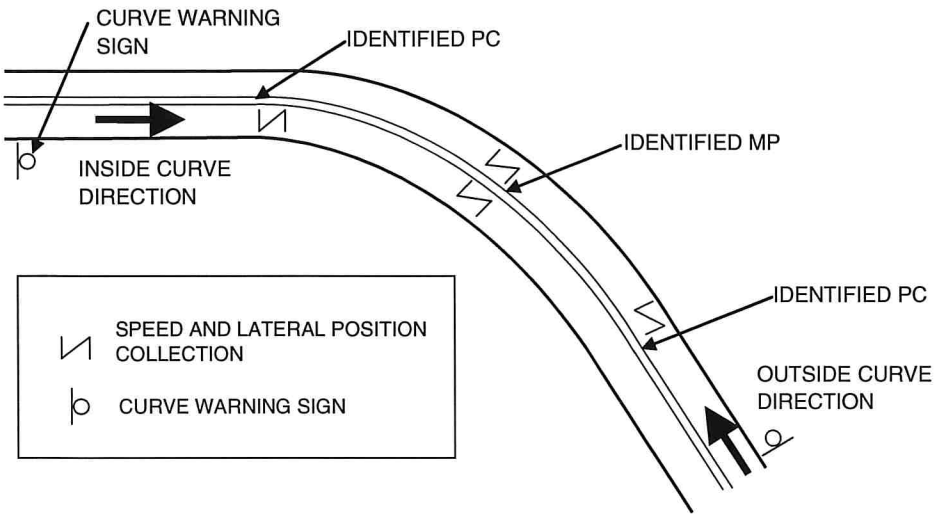


FIGURE 2 Data collection layout and location diagram.

- Allow for a minimum 10-day acclimation period,
- Collect data for the first after evaluation at both sites,
- Remove full-post chevrons from Site 1 and place at Site 2,
- Repeat the 10-day acclimation period, and
- Collect the second after evaluation at both sites.

## Data Processing and Screening

The raw vehicle data required processing and screening before the analysis. A software program quickly generated the vehicle speed data, but the lateral position data required a customized spreadsheet. The spreadsheet distinguished an individual vehicle passing along all three sensors at a data collection point. Software generated vehicle length, number of axles, and lateral position data. Research personnel ensured reliable data by removing any erroneous data points that the spreadsheet was unable to detect. Erroneous data points included vehicles with a speed of zero, improbable axle spacing, and a lateral position measurement that was greater than the sensor length.

The free-flowing screening process identified uninhibited vehicles and removed constrained vehicles. The process isolated the effects of the treatments on the vehicles and minimized the influence of lead vehicles. A driver traveling behind a slower moving vehicle may not be traveling at his or her preferred free-flow speed. A previous study identified that speeds of consecutive vehicles on two-lane rural highways were significantly different when there was a headway of 7 s or more between the lead and following vehicle (*10*). The screening process removed vehicles when there was a headway of 6 s or less between the following and the lead vehicle.

The vehicle type classification separated passenger vehicles and heavy vehicles into different lists. The vehicle operations of heavy vehicles and passenger vehicles are distinctively different. Heavy vehicles were identified by having more than two axles or exhibiting single axle spacing greater than 15 ft in length (*11*). The heavy vehicle percentages remained reasonably constant throughout the study, which indicated that vehicle patterns did not change considerably between the evaluation scenarios.

The time classification grouped vehicles into nighttime and daytime periods. The nighttime period referred to the hours that were devoid of natural sunlight, and the daytime period consisted of hours with ample sunlight. The nighttime period occurred between the hours of 9:00 p.m. and 6:00 a.m., and the daytime period occurred between 8:00 a.m. and 5:00 p.m. Also, the time classification minimized vehicles arriving in twilight. Vision can be hindered from the glare of the sun during twilight, and the period is associated with high crash rates (*12*). Although twilight is an appealing period for research, it was not evaluated because of the high variability in traffic patterns between evaluation scenarios. The before evaluation took place in the fall when there was an early sunset. The first and second after evaluations occurred in the spring when the sunset was later. Results could be unbalanced if the twilight period in the before evaluation included work commuters and after evaluations did not. Therefore, the time classification process minimized vehicles arriving in twilight by removing vehicles 30 min before and after sunrise and vehicles arriving 30 min before and after sunset.

Encroachment data required further processing. Lane line encroachments occurred when the outside edge of a vehicle's tire intruded on an edge line or centerline. The functional formatting derived the encroachment rates from the lateral position data and lane widths. The encroachment rates were expressed as a percentage of observed encroachments out of the total number of vehicles. Edge-line encroachment rates were easily established because lateral position measurements were collected from the outside edge of a vehicle's right tire. Centerline encroachment rates could not be directly obtained for each individual vehicle, so they were approximated by two aggregated track width values.

Researchers approximated centerline encroachment rates by assigning an 80- and 61-in. track width to all vehicles. The 80-in. track width was the maximum value from a list of 45 common large commercial sports utility vehicles, vans, and trucks. The larger and more conservative track width would account for most of the possible centerline encroachments. Any beneficial reduction in centerline encroachments would not be missed as a result of the large track width. The 61-in. track width was the average of 14 common passenger vehicles, such as a Toyota Camry and Honda Accord. All vehicle data were acquired from the manufacturers' websites, and heavy vehicles were excluded from the encroachment analysis.

Ultimately, the three data collection scenarios and formatting process yielded approximately 41,000 usable data points for the statistical analysis. The number of data points varied between the curve

locations, study sites, and evaluation scenarios. The data collection determined that the average daily traffic was about 1,000 vehicles for Site 1 and 1,500 vehicles for Site 2. The roadway volumes yielded data sets that ranged from 1,000 to 3,000 vehicles per PC and MP curve location for a given evaluation scenario. The percentages of heavy vehicles were consistently about 8% for Site 1 and 25% for Site 2 for each evaluation. Vehicles arriving during the nighttime period equated roughly to 10% to 15% of the daily traffic for each evaluation. Adverse weather was not an issue in any of the data collection scenarios, and no data points were removed because of weather. Overall, researchers deemed the sample set to be adequate.

### Analysis Methods

The data analysis used common statistical techniques to achieve the study objective. The statistical methods provided legitimacy and validity to the findings. The basic testing hypothesis stated that if findings were significant, then the null hypothesis was rejected and the alternative hypothesis was accepted. The null and alternative hypothesis tests were defined as follows:

- Null hypothesis ($H_o$): the tested variable or interaction failed to produce a significant difference between means.
- Alternative hypothesis ($H_a$): the tested variable or interaction produced a significant difference between means.

A considerable amount of the findings came from evaluating the means of vehicle data. The multivariate analysis of variance (MANOVA) tested for the differences between mean values of multiple populations as a function of independent variables and interactions between the independent variables (*13*). The dependent variables in the model were speed and lateral position data, and the independent variables were

- Site (Site 1 and Site 2),
- Curve location (PC and MP),
- Curve direction (inside and outside),
- Time (nighttime and daytime),
- Vehicle type (passenger vehicle and heavy vehicle), and
- Treatment (baseline, chevrons, and full-post chevrons).

The overall MANOVA showed significant main effects for variables at the .05 level. See Ré for further details on the full statistical analysis (*14*). Along with the MANOVA models, Tukey's honestly significant difference (HSD) tested for significance among the treatment means. Tukey's HSD is a paired comparison that compares the means of subgroups in the MANOVA model and is similar in manner to the *T*-test. The *Z*-test was used to compare proportions (rates) of two samples. The *Z*-test identified whether there was a significant difference in the encroachment rates or high speed comparison. The *F*-test assessed whether the speed and lateral position standard deviation values were significantly different.

## FINDINGS

Researchers applied the data analysis methods for vehicle speed and for lateral position data. The lateral position findings are presented first followed by the speed findings. Analysis of both vehicle response variables allowed this study to determine whether full-post chevrons achieved additional benefits over standard chevrons. A more detailed and rigorous assessment of the findings is found in Chrysler et al. (*8*) and Ré (*14*).

### Lateral Position Findings

In the analysis a MANOVA test was performed on the lateral position data for the overall data set and for Site 1 and Site 2. Most important, the MANOVA test determined that the main effects of the treatment variable were significant in all three models. Curve location, curve direction, and time classification were also significant variables. The model revealed that vehicles were inclined to move closer to the centerline on an outside curve and shift closer to the edge line on an inside curve. This shift has been observed in past studies and is referred to as curve flattening. The magnitude of the shift was more pronounced at the MP than at the PC. For the time classification, drivers tended to be situated closer to the centerline during the nighttime period regardless of curve direction. The average nighttime lateral position value was 2.36 in. closer to the centerline than in the daytime. In the model, the main effects of the vehicle type variable and the interaction between vehicle type and treatments were not statistically significant. The researchers reasoned that passenger vehicles and heavy vehicles exhibited similar lateral position traits and the treatments achieved a significant effect for both vehicle types.

Table 1 shows the results from Tukey's HSD test for the overall data set and for Site 1 and Site 2. Values in the table are the mean distances from the centerline to the outside edge of a vehicle's right tire. The travel lane width for each curve location is about 120 in. All mean values for chevrons and full-post chevrons were significantly different from the baseline values, which verified that both treatments had a considerable effect on vehicle lateral position. In a treatment comparison, chevrons and full-post chevrons mean values were not significantly different in the overall and Site 2 tests. The treatment means in the Site 1 test were significantly different, but the difference was less than 1 in. Chevrons and full-post chevrons moved vehicles away from the centerline and closer to the edge line for the inside and outside curve directions. There were some differences between daytime and nighttime treatment results, but there were no substantial disparities and the majority of the differences remained about 2 in. On average, both chevrons and full-post chevrons moved vehicles away from the centerline by approximately 15 in. from the baseline evaluation.

In the lateral position standard deviation comparison, all treatment values were lower than the baseline values. The reduction in standard deviation values indicated that both treatments achieved more uniform and consistent lane position at both the PC and MP locations. The overall standard deviation values for the baseline evaluation, chevrons, and full-post chevrons were 13.55, 7.52, and 7.86 in., respectively. The *F*-test proved that chevrons and full-post chevrons produced significantly lower standard deviation values. In

TABLE 1 Tukey's HSD Mean Lateral Position Results

| | Lateral Position (in.) | | |
|---|---|---|---|
| Treatment | Site 1 | Site 2 | Overall |
| Baseline | 89.13 *a* | 85.28 *a* | 87.12 *a* |
| Chevrons | 103.60 *c* | 101.36 *b* | 102.59 *b* |
| Full-post chevrons | 102.89 *b* | 101.61 *b* | 102.26 *b* |

NOTE: Means followed by the same letter are not significantly different according to MANOVA and Tukey's HSD test.

a treatment comparison, there was only one *F*-test in which values for chevrons and full-post chevrons were significantly different.

The more uniform and consistent lane position was reflected in the encroachment rates. Chevrons and full-post chevrons reduced the centerline encroachment rates for the 80- and 61-in. vehicle track width. On average, chevrons reduced the centerline 80-in. encroachment rates by approximately 93% and full-post chevrons reduced rates by 88%. Both treatments lowered the centerline encroachments for a 61-in. track vehicle to approximately zero in several instances. The *Z*-test determined that all centerline encroachment rates for chevrons and full-post chevrons were statistically different from the baseline rates for both the 80- and 61-in. vehicle track widths. Most of the edge-line encroachments were lowered slightly or remained approximately unchanged. In a treatment comparison, there were only two tests in which chevrons and full-post chevrons produced significantly different rates. Overall, chevrons and full-post chevrons achieved similar and beneficial encroachment results.

### Speed Findings

The MANOVA test determined that the main effects of the treatments produced significantly different speed results for the overall data set and at Site 1 and Site 2. Vehicle type, curve location, and curve direction were also significant variables. The main effects of the time variable were not significant for Site 2, which indicated that vehicle speeds were similar in the nighttime and daytime periods at this specific curve. In the overall test, the interaction between time and the treatment was not significant. This suggested that the treatments achieved a similar change in speed for nighttime and daytime periods.

Table 2 shows the mean speed values from Tukey's HSD test for the overall data set and for Site 1 and Site 2. Both chevrons and full-post chevrons achieved significantly lower mean speeds than in the baseline evaluation. In the overall test, chevrons and full-post chevrons produced a speed reduction of 1.28 and 2.20 mph, respectively. Mean speed consistently decreased as the treatments were added. The mean speed values for full-post chevrons were closest to the alternate advisory curve speeds, which are 55 mph for Site 1 and 50 mph for Site 2. Despite the significantly different test results, the mean speed reductions were inconsequential.

The speed standard deviation findings had a reverse trend from most of the previous findings. Many of the standard deviation values for chevrons and full-post chevrons increased from the baseline evaluation. The average standard deviation values increased from 7.80 mph in the baseline evaluation to 8.25 mph for chevrons and 8.36 mph for full-post chevrons. The *F*-test concluded that most of the standard deviation values for both chevron treatments were not significantly different from the baseline standard deviation. In a treatment comparison between chevrons and full-post chevrons, none of the standard deviation values were significantly different.

**TABLE 2 Tukey's HSD Mean Speed Results**

| | Speed (mph) | | |
|---|---|---|---|
| Treatment | Site 1 | Site 2 | Overall |
| Baseline | 58.28 *a* | 54.79 *a* | 56.46 *a* |
| Chevrons | 56.77 *b* | 53.24 *b* | 55.18 *b* |
| Full-post chevrons | 56.32 *c* | 52.14 *c* | 54.26 *c* |

NOTE: Means followed by the same letter are not significantly different according to MANOVA and Tukey's HSD test.

Researchers reasoned that the increase in speed standard deviation values by both chevron treatments was not substantial.

Tables 3 and 4 compare the alternate advisory curve speeds with the measured high vehicle speeds. The high vehicle speeds include the 85th percentile speed and the percentage of vehicles exceeding 60, 65, and 70 mph at the test sites. Similar to the mean speed analysis, 85th percentile speed values for chevrons and full-post chevrons were lower than the baseline values. On average, full-post chevrons lowered the baseline 85th percentile speed by 2.2 mph and chevrons lowered it by 1.3 mph. Both chevron treatments reduced the percentage of vehicles exceeding speeds of 60, 65, and 70 mph. The *Z*-test proved that the chevron and full-post chevron reductions were significantly lower than the percentages in the baseline evaluation. In a treatment comparison, full-post chevrons significantly reduced high speed percentages further than chevrons in all but one test. Overall, full-post chevrons consistently reduced the high vehicle speeds further than chevrons, but the additional reduction was again considered not to be substantial.

## CONCLUSIONS AND RECOMMENDATIONS

Researchers summarized lateral position and speed findings for chevrons and full-post chevrons. Substantial and negligible differences between chevron treatments were addressed. This study closes with final comments and treatment recommendations for full-post chevrons.

### Lateral Position Conclusions

Findings determined that chevrons and full-post chevrons achieved beneficial changes in lateral position MOEs. Both chevron treatments consistently achieved significantly different results from the baseline evaluation. Chevrons and full-post chevrons moved vehicles away from the centerline and from oncoming traffic by approximately 15 in. The baseline lateral position standard deviation was nearly cut in half from 13.55 in. to 7.52 in. for chevrons and to 7.86 in. for full-post chevrons. Both chevron treatments reduced estimated centerline

**TABLE 3 Advisory Curve Speeds**

| | Site 1 | | | Site 2 | | |
|---|---|---|---|---|---|---|
| Speed Parameter | Baseline | Chevrons | Full-Post Chevrons | Baseline | Chevrons | Full-Post Chevrons |
| Posted advisory speed (mph) | | 45 | | | 50 | |
| Alt. advisory curve speed (mph) | | 55 | | | 50 | |
| 85th percentile speed (mph) | 65.72 | 64.17 | 63.8 | 64.17 | 63.06 | 61.91 |

NOTE: Alt. = alternate.

TABLE 4 Measured High Vehicle Curve Speeds

| | Percentage of Vehicles Exceeding a Given Speed | | | | | |
|---|---|---|---|---|---|---|
| | Site 1 | | | Site 2 | | |
| Speed (mph) | Baseline | Chevrons | Full-Post Chevrons | Baseline | Chevrons | Full-Post Chevrons |
| 60 | 43 | 34 | 33 | 32 | 25 | 21 |
| 65 | 17 | 13 | 11 | 13 | 10 | 8 |
| 70 | 4 | 3 | 2 | 4 | 3 | 2 |

encroachments by approximately 90%, and edge-line encroachments remained relatively unchanged. Results for chevrons and full-post chevrons were quite similar. In the overall data set, the difference in mean lateral position values between the chevron treatments was 0.33 in. Most of the mean and standard deviation statistical comparisons were also not significantly different. Both chevron treatments were effective in achieving more uniform and beneficial lateral position results, but full-post chevrons did not yield substantial gains over chevrons.

### Speed Conclusions

Findings established that chevrons and full-post chevrons produced significant changes in speed MOEs. Tukey's HSD test identified that both chevron treatments significantly reduced mean speed values closer to the alternate advisory curve speed from the baseline evaluation. Results showed that chevrons and full-post chevrons slightly increased the speed standard deviation values from the baseline evaluation. The average standard deviation values increased from 7.80 mph in the baseline evaluation to 8.25 mph for chevrons and to 8.36 mph for full-post chevrons, which was deemed acceptable.

In a treatment comparison, most of the tests between chevrons and full-post chevrons were significantly different. Full-post chevrons consistently produced mean speed values that were closer to the alternate advisory curve speed. In the overall data set, chevrons lowered the mean speed by 1.28 mph from the baseline evaluation, and full-post chevrons achieved a reduction of 2.20 mph. Although results for full-post chevrons were significantly lower, the 0.92-mph further reduction was considered inconsequential. Researchers concluded that full-post chevrons did not achieve substantial benefits in vehicle speed over chevrons.

### Recommendations

In a treatment comparison, experimental full-post chevrons did not produce substantial gains over standard chevrons. Agencies using full-post chevrons in a similar roadway situation may not experience substantial benefits over standard chevrons. Although full-post chevrons did not achieve incremental benefits, the findings did not indicate that the treatment had a detrimental effect on vehicle operations. Full-post chevrons may have value in situations that are different from those evaluated in this study. The current MUTCD language allows agencies to implement full-post chevrons as an option, at their discretion. Researchers recommend that the MUTCD should continue to present full-post chevrons as an optional delineation tool. On the basis of this research, researchers do not recommend any changes to the MUTCD.

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*The Traffic Control Devices Committee peer-reviewed this paper.*

# Protected–Permissive Left-Turn Signal Control Mode

## New Analytical Approach to Estimate Operational Benefit and Safety Cost

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**Protected–permissive left-turn (PPLT) and protected only (PO) left-turn signal control modes have been widely used at signalized intersections. The selection between these two control modes is a complicated process in which practitioners need to consider safety and operational impacts. In this study researchers developed analytical models for estimating the operational benefits and the safety risks associated with the use of the PPLT control mode. Evaluation results demonstrated that the proposed models could provide accurate and reliable estimates for the delay reduction and the potential conflicts caused by the use of the PPLT mode. Results of this study will be useful for developing quantitative criteria for the selection between the PO and PPLT modes.**

The left-turn operation is a critical component for the safe and efficient operation of a signalized intersection. The proper type of left-turn phasing will result in reduced traffic delay, improved traffic flow, and decreased accident rates (*1, 2*). Since the introduction of the protected–permissive left-turn (PPLT) signal control mode in the 1920s, PPLT has been used more often to accommodate left-turn movements at signalized intersections because it increases left-turn capacity by providing a protected turn phase as well as a permissive phase during which left turns can be made as opposing traffic allows (*3*). However, safety is a major concern in the implementation of the PPLT control. Under certain circumstances, such as heavy opposing volume, high-speed approaches, and restricted sight distance, it may not be safe to make permissive left turns. Therefore, the selection of an appropriate left-turn phasing treatment is a complicated process in which a trade-off between safety and operational efficiency exists. This study was done to develop models for quantifying the benefit and cost of the PPLT control mode compared with the protected only (PO) mode.

Previous studies have developed warrants or guidelines for selecting proper left-turn control modes, such as PO and PPLT, for signalized intersections. Most of these warrants or guidelines have been developed on the basis of engineering judgments instead of systematic intersection safety and operation analyses (*4*). Some researchers have conducted intersection safety analyses on the basis of historical accident data and operational analyses on the basis of traffic delay data collected from the field (*5, 6*). The problems with the historical accident data-based safety analyses are the unreliability of accident records and the long period of time required to achieve adequate sample sizes. The problem with the field data–based operational analyses is the high cost in collecting accurate traffic delay data in the field. Some researchers have used traffic simulation approaches to analyze intersection operational performance under different types of left-turn signal control modes. However, the cost of coding, calibrating, and running traffic simulation models is also high. To address these problems, in this study researchers developed analytical models for estimating the delay reduction and the safety risk associated with the use of the PPLT control mode. The model for estimating the delay reduction was developed by estimating the available gaps during the permissive left-turn phase on the basis of the queuing diagram delay estimation method. For the safety risk analysis, an analytical model for estimating the potential conflicts during the permissive left-turn phase is developed by calculating the probability that a left-turn vehicle will take a relatively risky (small) gap in the opposing traffic to make a permissive turn. Finally, to evaluate the accuracy of these two analytical models, researchers compared the model results with the actual data collected from the study intersections and the results of some traditional methods. In addition, on the basis of the developed models, researchers proposed a framework for conducting the benefit and cost analyses of using the PPLT mode. Results of this study will allow traffic engineers to analyze the benefit and cost of using the PPLT mode at any intersection with basic intersection information. It will also be useful for developing quantitative guidelines for selecting between the PO and PPLT modes.

Following a brief review of the existing studies on the selection criteria for the PPLT signal control mode, this paper details the development of the models for estimating the delay reduction and the safety risk caused by using the PPLT signal control mode. After that, a framework is proposed for selecting between the PO and PPLT signal control mode. Finally, a case study is provided to demonstrate the application of the proposed models.

## LITERATURE REVIEW

Numerous studies have been conducted on the development of warrants for the PPLT signal control mode. Most warrants are for updating permissive-only operation to the PPLT or PO operation.

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Some of them also provide guidelines for selecting between the PPLT and PO left-turn signal control modes. Most of these guidelines use volume-based warrants, in which a constant threshold is set for the left-turn volume or for the cross product of left-turn and opposing through (OTH) volumes. Upchurch collected delay data from four intersections with PPLT and PO left-turn phasing and found the threshold volumes (including left-turn volume, opposing volume, and their cross product) at which the delay of PPLT mode was not significantly lower than that of the PO mode (*7*).

Zhang et al. developed variable thresholds for the left-turn volume or the cross product of the left-turn and opposing volumes according to the results of traffic simulations (*8*). These volume-based criteria were developed on the basis of only intersection delay analyses (using traffic simulation models), and the safety effects of the PPLT mode have not been well considered in the development of these volume criteria. Actually, with the increase in traffic volume, it will become more difficult for left-turning vehicles to find adequate gaps in the opposing traffic to make safe permissive left turns. Thus, safety is the major reason for not allowing permissive left turns under heavy traffic volume conditions.

Other research efforts have developed warrants for the PPLT control mode on the basis of intersection accident rates or conflict rates. Cottrell collected historical accident data from intersections with the PPLT control mode and found a threshold value for the cross product of the left-turn and opposing traffic volumes (*3*). Intersections at which traffic volume is beyond this threshold have indicated significantly higher accident rates.

Some researchers have developed warrants on selecting between PPLT and PO on the basis of both benefit and cost analyses. Agent collected delay and accident data from four intersections that changed their left-turn signal control mode from PO to PPLT (*9, 10*). It was found that the benefit and cost ratio was more than one at all locations. Thus, the PPLT mode was recommended for these intersections. It was also found that when opposing volume is greater than 1,000 vehicles per hour (veh/h), the operational benefit of the PPLT mode is very low.

Benham proposed an analytical method to derive the left-turn capacity during the permissive phase by calculating the available gaps in the opposing traffic on the basis of the assumption that the arrival of the through vehicles follows a Poisson distribution (*11*). The model developed by Benham, however, has the following limitations. First, it does not consider the effects of the initial opposing queue and the different types of signal phase sequences on the left-turn delay. In addition, the Poisson arrival assumption cannot be met when the traffic volume is close to capacity. These problems were addressed by the delay model developed in the current study.

Zhang et al. proposed to measure the safety risk of permissive left-turn movement by calculating the probability that a left-turning vehicle will take a relatively risky (small) gap in the opposing traffic to make a permissive turn (*12*). However, the model developed by Zhang was used mainly for analyzing the safety risk of permissive-only left turn signal control and cannot be applied directly for the PPLT signal control mode.

In this study the permissive left-turn capacity model proposed by Benham and the safety risk model proposal by Zhang were modified according to the special features of the PPLT operation. In addition, the current study used a queuing diagram method to develop the model for estimating the left-turn delay reduction associated with the implementation of the PPLT control mode.

## DATA COLLECTION

Field data were collected from 10 approaches with a PPLT left-turn control at four different intersections in Austin, Texas: (*a*) Intersection #103, Lamar at 38th; (*b*) Intersection #855, Loop 360 at Stonelake, (*c*) Intersection #170, 51st at Airport, and (*d*) Intersection #5, Lamar at Koenig. All intersection approaches had quite heavy left-turn and opposing traffic volumes.

Traffic video was recorded at these study intersections for 2 to 3 h (during either the a.m. peak hours or the p.m. peak hours) and processed in the lab. For each of the approaches the following general data were collected:

1. Traffic flow–related data, including opposing through volume and left-turn volume;
2. Traffic signal-timing plan (a signal-timing diagram has been provided for each intersection with the detailed timing of each signal phase); and
3. Geometric conditions, including the number of left-turn lanes, the number of opposing lanes, and the alignment of the intersection.

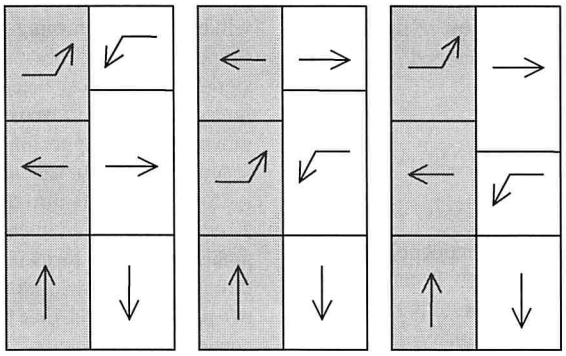
In addition, the following data were collected especially for the model evaluation purpose:

1. Number of left turns during the permissive phase. This was obtained by directly observing the traffic video.
2. Number of potential conflicts. Potential conflict is the event in which a left-turn vehicle makes a permissive left-turn in a gap less than a certain threshold. In this study this threshold was set as the time needed for a left-turning vehicle to clear the intersection, which was estimated on the basis of the intersection's geometric conditions, average car length, and average acceleration rate. The potential conflict data were collected by observing the collected traffic video with the assistance of a Microsoft Excel microprogram, which was used to quickly record the time that each vehicle passed the conflict points.

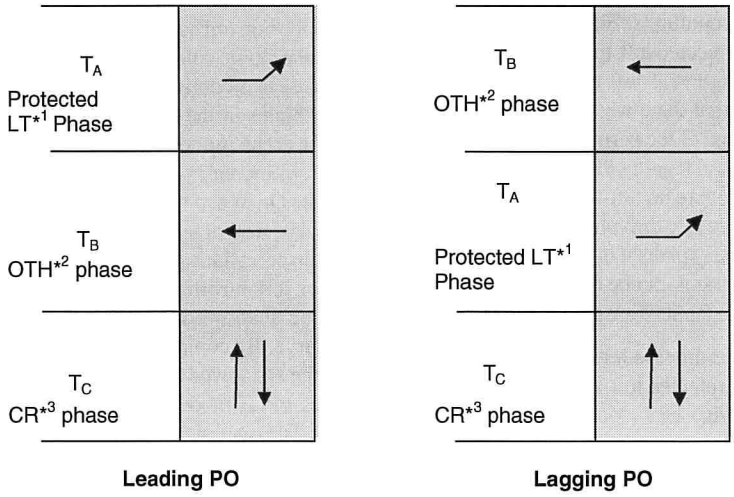
These two types of data were used for evaluating the models developed for estimating the operational benefits (the number of left turns during the permissive phase) and the safety cost (the number of potential conflicts during the permissive phase) associated with the implementation of the PPLT signal control mode.

## METHODOLOGY

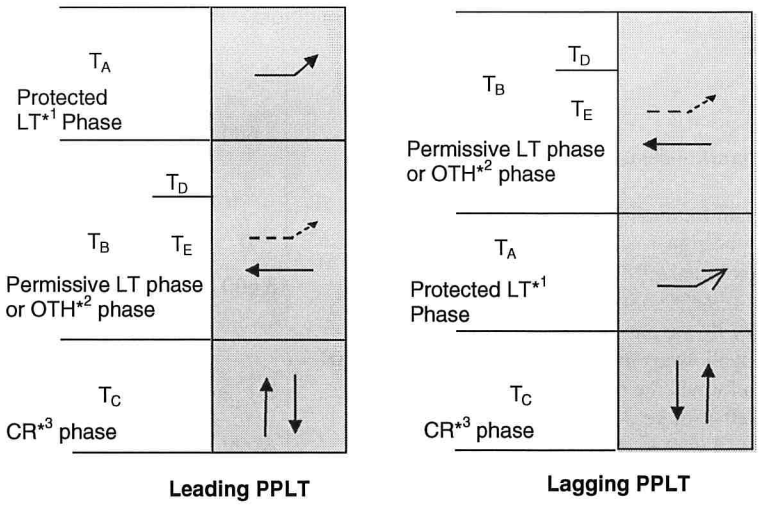
To analyze the benefits and costs of the PPLT model, left-turn operations under the PO and PPLT modes need to be compared (*4*). The signal-timing diagrams for the PO and PPLT signal control mode are presented in Figures 1*b* and 1*c*. There are three different types of signal phasing sequences as presented in Figure 1*a*. However, for one direction, there are only two types of sequences, leading or lagging left turn as shown in Figures 1*b* and 1*c*. From these figures, it can be seen that the major difference between PPLT and PO control mode is that besides the protected left-turn phase, PPLT mode has a permissive left-turn phase, during which the left-turning vehicles are allowed to make left turns if there are large enough gaps in the opposing traffic. This permissive phase in the PPLT mode can increase the left-turn capacity of the intersection approach, which will reduce the left-turn traffic delay. However, safety risks will also increase with the use of a permissive phase because drivers of the left-turning vehicles may misjudge the gaps in the opposing traffic, which may result in accidents. Therefore, the benefit that the PPLT




(a)




(b)




(c)

FIGURE 1 Signal timing diagrams for PO and PPLT signal control modes: (*a*) three different types of left-turn-signal phasing sequences, (*b*) signal timing diagrams for PO left-turn signal control mode, and (*c*) signal timing diagrams for PPLT signal control mode (LT*1 is the left-turn movement, OTH*2 is the opposing through movement, and CR*3 is the cross movement).

mode has over the PO mode is the reduction of left-turn traffic delay, and the cost is the increase in safety risks (*13*). In the following sections, models for quantifying the benefit and cost of the PPLT control mode compared with the PO mode are developed.

## Benefit Analysis

### *Methodology*

Because the benefit of the PPLT control mode is the reduction of left-turn delay, the left-turn delay under the PO and the PPLT modes were estimated and compared in the current study. In this study the delay of the left-turn traffic at the intersections with different types of traffic signal control mode, signal phasing sequence, and traffic volume conditions was estimated using the queuing diagram method (*14, 15*).

There are six queuing diagrams presented in Figures 2 and 3 resulting from different combinations of left-turn signal control modes, phasing sequences, and traffic conditions. Figures 2*a* and 3*a* show leading protected and lagging protected left-turn control. Figures 2*b* and 3*b* show leading and lagging PPLT left-turn control with high left-turn volume, which means that there were some left-turn vehicles waiting in the queue at the end of the permissive left-turn phase. Figures 2*c* and 3*c* show leading and lagging PPLT with low left-turn volume, which means that there were no left-turning vehicles in the queue at the end of the permissive left-turn phase. The queuing diagrams in Figures 2*a* and 2*b* will be explained in detail in the following sections, and other queuing diagrams can be explained similarly.

The following symbols and assumptions are used in Figures 2 and 3:

$S_{per}$ = saturation flow of the permissive left-turn phase in veh/h,
$S_{pro}$ = saturation flow of the protected left-turn phase in veh/h,
$Q_L$ = left-turn volume in veh/h,
$T_A$ = protected left-turn phase,
$T_B$ = permissive left-turn phase or the opposing through phase,
$T_C$ = cross-movement phase,
$T_D$ = portion of $T_B$ used to discharge the initial opposing through queue,
$T_E$ = effective green time for left-turn vehicles to make permissive left turns, and
$T_E = T_B - T_D$
(see Figure 1).

It is assumed that the intersections are under unsaturated conditions. So, $S_{pro} > Q_L$.

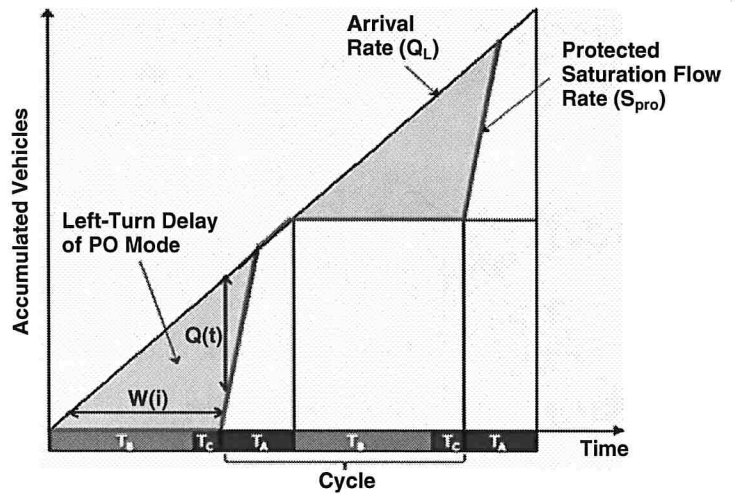
Figure 2*a* is the queuing diagram for leading PO left-turn control mode. As shown in this figure, one signal cycle consisted of three phases: (*a*) $T_A$, the phase for the protected left-turn movement; (*b*) $T_B$, the phase for opposing through movement, with no left-turn movement allowed during this phase; and (*c*) $T_C$, the phase for the cross movement, with no left-turn movement allowed during this phase. In this case the left-turn queue begins to form at the beginning of phase $T_B$. During phases $T_B$ and $T_C$, the left-turn vehicles arrive at rate $Q_L$ (the left-turn volume in veh/h) and wait in the queue at the intersection. Thus, in these two phases, the departure curve (blue color line) remains at the same level but the arrival curve increases continually at rate $Q_L$. During the protected left-turn phase, $T_A$, left-turn vehicles in the queue were discharged at rate $S_{pro}$. Thus, the departure curve rises and catches up with the arrival curve at a certain point during phase $T_A$. Therefore, the aggregate delay (vehicle * hours) for left-turning vehicles per cycle can be calculated as the area of the triangle (the shadowed area) between the arrival curve and the departure curve.

Figure 2*b* is the queuing diagram for leading PPLT left-turn control mode with high left-turn volume. Here, the high left-turn volume means that the left-turn saturation flow during the permissive phase is less than the left-turn arrival rate, that is, $S_{per} < Q_L$. As a result, there will be some left-turning vehicles waiting in the queue at the end of the permissive left-turn phase. For the PPLT control mode, the whole signal cycle also consisted of three phases: (*a*) $T_A$, the phase for the protected left-turn movement, (*b*) $T_B$, the phase for permissive left-turn movement with opposing through movement, and (*c*) $T_C$, the phase for cross movement with no left-turn movement allowed in this phase. As before, the left-turn queue will be accumulated during phases $T_B$ and $T_C$. However, in phase $T_B$, some left-turning vehicles can still make permissive turns if there are adequate gaps in the opposing through traffic flow. Actually, this phase can be further divided into two parts: (*a*) $T_D$, the time for discharging the initial opposing through queue at the beginning of the $T_B$, during which no left-turning vehicles could make permissive turns, and (*b*) $T_E$, the effective green time for permissive left-turn movement. The initial opposing queue will be cleared during $T_D$, and after that the opposing vehicles will arrive at the intersection randomly and will make permissive left turns as opposing traffic allows during $T_E$. Therefore, during $T_D$, the departure curve (blue color line) remains at the same level but the arrival curve increases continually at rate $Q_L$. When phase $T_E$ begins, left-turning vehicles in the queue are discharged at rate $S_{per}$. After that, during the cross-movement phase, $T_C$, the departure curve (blue color line) stops increasing again and the arrival curve increases continually at rate $Q_L$. When the protected left-turn phase, $T_A$, begins, the departure curve rises at rate $S_{pro}$ and will catch up with the arrival curve at a certain point during phase $T_A$. Thus, the aggregate delay (vehicle * hours) for left-turning vehicles per cycle can be calculated as the area (the shadowed area) between the arrival curve and the departure curve. By comparing the left-turn delay under the PO control mode (the shadowed areas in Figure 2*a*), the total delay reduced by using the PPLT mode can be derived, which is the dark area in Figure 2*b*.

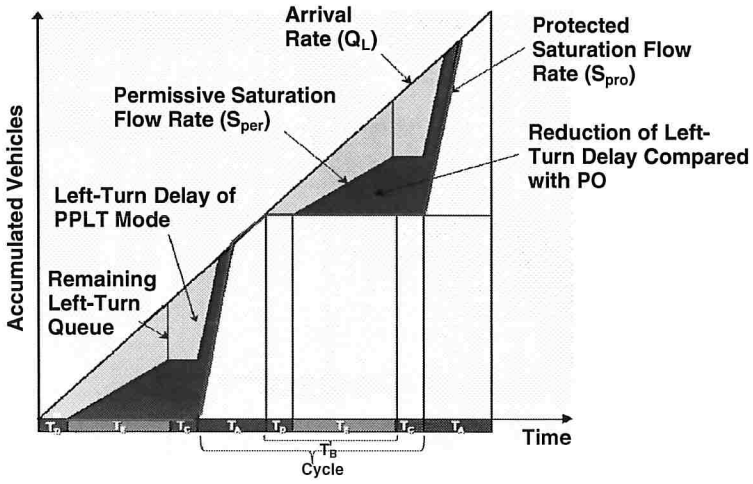
The queuing diagrams for other situations presented in Figures 2 and 3 can be explained similarly. In Figures 2*b*, 2*c*, 3*b*, and 3*c*, the dark gray areas represent the delay reduction of the PPLT control mode compared with the PO mode under four different conditions (different types of signal phasing sequence and traffic volume conditions). These four areas can be calculated using Equation 1.

$$\Delta D = \begin{cases} \dfrac{\frac{1}{2}S_{per}}{3{,}600} * T_E\left(T_E + 2T_C\right) + \dfrac{2T_E S_{per} Q_L\left(T_B + T_C\right) - S_{per}^2 T_E^2}{7{,}200\left(S_{pro} - Q_L\right)} & \text{if the queuing diagram is Figure 2}b \\ \dfrac{\frac{1}{2}Q_L}{3{,}600} * \left[\dfrac{S_{pro}\left(T_B + T_C\right)^2}{S_{pro} - Q_L} - \dfrac{S_{pro}T_C^2}{S_{pro} - Q_L} - \dfrac{S_{per}T_D^2}{S_{per} - Q_L}\right] & \text{if the queuing diagram is Figure 2}c \\ \dfrac{\frac{1}{2}S_{per}}{3{,}600} * T_E^2 + \dfrac{2T_E S_{per} Q_L\left(T_B + T_C\right) - S_{per}^2 T_E^2}{7{,}200\left(S_{pro} - Q_L\right)} & \text{if the queuing diagram is Figure 3}b \\ \dfrac{\frac{1}{2}Q_L}{3{,}600} * \left[\dfrac{S_{pro}\left(T_B + T_C\right)^2}{S_{pro} - Q_L} - \dfrac{S_{per}\left(T_D + T_C\right)^2}{S_{per} - Q_L}\right] & \text{if the queuing diagram is Figure 3}c \end{cases} \quad (1)$$

where $\Delta D$ is the delay reduction of the PPLT mode per cycle compared with PO mode.




(a)

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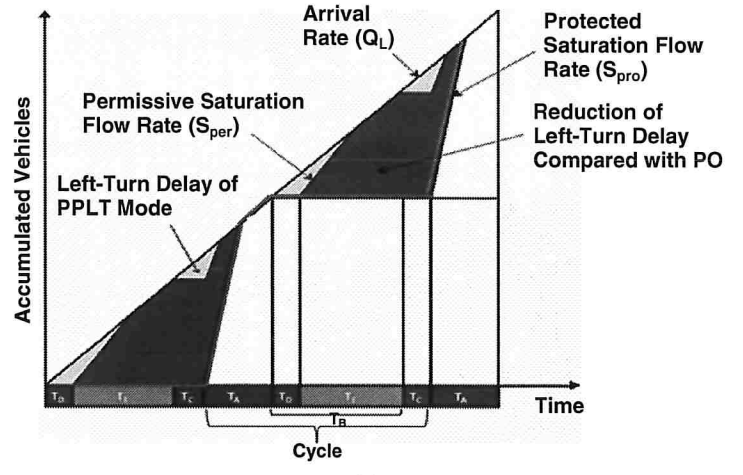
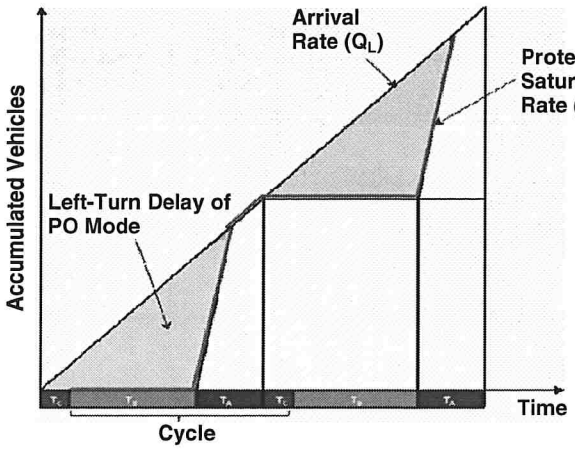
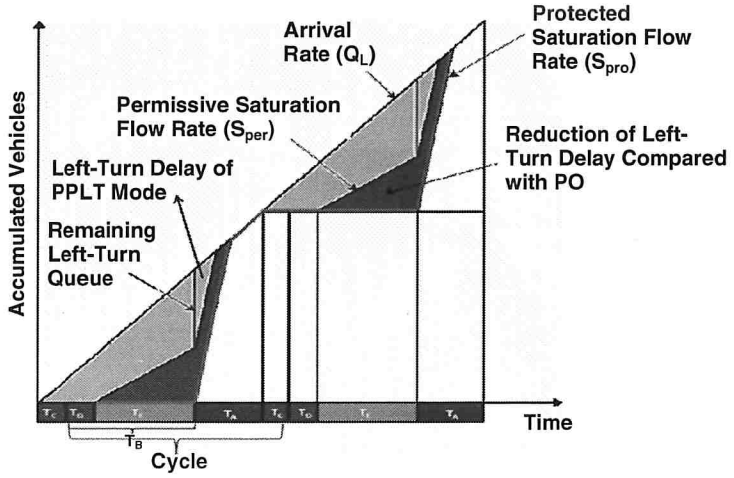



(c)

FIGURE 2 Queuing diagrams for leading left-turn control: (*a*) leading PO left-turn control, (*b*) leading PPLT left-turn control with high left-turn volume, and (*c*) leading PPLT left-turn control with low left-turn volume.




(a)

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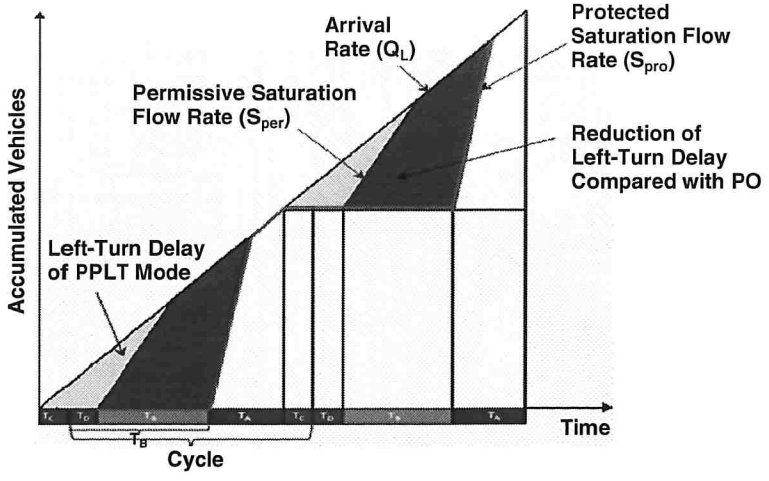



(c)

FIGURE 3 Queuing diagrams for lagging left-turn control: (*a*) lagging PO left-turn control, (*b*) lagging PPLT left-turn control with high left-turn volume, and (*c*) lagging PPLT left-turn control with low left-turn volume.

**Determine the Values of Variables in Equation 1** To estimate the delay reduction of using the PPLT mode, the values of eight variables in Equation 1 need to be determined first. These variables were grouped into two categories. The first category of variables consists of those related to a signal-timing plan. These variables include $T_A$, $T_B$, $T_C$, $T_D$, and $T_E$. Among these variables, the value of $T_A$, $T_B$, $T_C$ can be directly obtained according to the signal-timing plan. $T_E$ and $T_D$ are related ($T_E + T_D = T_B$). Thus, $T_E$ is determined by $T_D$, where $T_D$ is defined as the portion of $T_B$ used to discharge the initial opposing through queue. For the lead-lead or lag-lag signal phasing sequence, $T_D$ is equal to the total time used for discharging the opposing through vehicles that queued during phases $T_A$ and $T_C$, which is noted as $T_{opp}$. For the lead-lag signal phasing sequence, if the subject direction is the leading left-turn direction, then $T_D$ is still equal to $T_{opp}$. If the subject direction is the lagging left-turn direction, the opposing vehicles began to move at the beginning of phase $T_A$. Thus, $T_D$ is equal to $T_{opp} - T_A$ if all opposing queues could not be cleared by the end of $T_A$. Otherwise, $T_D$ is equal to zero. Thus, $T_D$ can be expressed by the following equation:

$$T_D = \begin{cases} \max(T_{opp} - T_A, 0) & \text{for the lagging left-turn direction in the lead-lag signal phasing} \\ T_{opp} & \text{others} \end{cases} \quad (2)$$

Because the initial opposing queue is accumulated during phases $T_A$ and $T_C$, $T_{opp}$ can be estimated by using the following equation:

$$T_{opp} = \frac{\left(\frac{T_A + T_C}{C} \times \frac{Q_O}{N_O} \times h\right)}{\frac{3{,}600}{C}} = \frac{(T_A + T_C)}{3{,}600} \times \frac{Q_O}{N_O} \times h \quad (3)$$

where

$Q_O$ = opposing volume in veh/h;
$C$ = cycle length;
$h$ = average headway of opposing through vehicles, assumed as 2 s; and
$N_O$ = number of opposing through lanes.

The second category of variables used in Equation 1 consists of the variables related to traffic flow. These variables include the saturation flow rate during the permissive left-turn movement ($S_{per}$), the saturation flow rate during the protected left-turn movement ($S_{pro}$), and the left-turn volume of the subject approach ($Q_L$). These variables will determine the slopes of the curves in the queuing diagrams. Of these variables, $S_{pro}$ can be estimated by assuming that the average headway of the left-turn vehicles during the protected phase is 2.1 s. $S_{per}$ can be estimated on the basis of the arrival pattern of the opposing traffic and the number of lanes in the opposing direction. The following section will detail the derivation of $S_{per}$.

**Derive Permissive Left-Turn Saturation Flow Rate, $S_{per}$** After the opposing queue is completely discharged, the left-turning vehicle will search for large gaps in the opposing traffic to make permissive left turns during the effective green time phase ($T_E$). It is assumed that the arrival of opposing vehicles follows a Poisson distribution. Thus, the probability that a gap in the opposing traffic is larger than a critical gap ($t$ s) can be calculated using the following equation:

$$P(\text{gaps} \geq t) = e^{-Q_O t/3{,}600} \quad (4)$$

where $t$ is the critical gap (s). The critical gap ($t$) is the gap that most drivers will take to make a permissive left turn. In this paper, $t$ is assumed to be equal to the left-turning maneuver time, which is denoted as $t_{LT}$. Note that $t_{LT}$ can be estimated on the basis of the intersection's geometric conditions, the average car length, and the average acceleration rate. The method proposed by Zhang et al. is used to calculate $t_{LT}$ (*12*).

On the basis of Equation 4, the number of gaps per hour that is larger than $t$ s can be derived as follows:

$$\text{number of } (\text{gaps} \geq t) = Q_O * P(\text{gaps} \geq t) = Q_O * e^{-Q_O t/3{,}600} \quad (5)$$

Similarly, the number of gaps within the range of [$t$, $2t$], [$2t$, $3t$], . . . , [$nt$, $(n + 1)t$] per hour can be derived from the following equations:

$$\text{number of } (t \leq \text{gaps} \leq 2t) = Q_O * (P(\text{gaps} \geq t) - P(\text{gaps} \geq 2t)) = Q_O\left(e^{-Q_O t/3{,}600} - e^{-Q_O 2t/3{,}600}\right) \quad (6)$$

$$\text{number of } (2t \leq \text{gaps} \leq 3t) = Q_O * (P(\text{gaps} \geq 2t) - P(\text{gaps} \geq 3t)) = Q_O * \left(e^{-Q_O 2t/3{,}600} - e^{-Q_O 3t/3{,}600}\right) \quad (7)$$

$$\text{number of } (nt \leq \text{gaps} \leq (n+1)t) = Q_O * (P(\text{gaps} \geq nt) - P(\text{gaps} \geq (n+1)t)) = Q_O * \left(e^{-Q_O nt/3{,}600} - e^{-Q_O (n+1)t/3{,}600}\right) \quad (8)$$

For the gaps that are within [$t$, $2t$], only one left-turn vehicle can clear the intersection. For the gaps that are within [$2t$, $3t$], two left-turn vehicles can clear the intersection. Generally, for the gaps that are within [$nt$, $(n + 1)t$], $n$ left-turn vehicles can clear the intersection. By summing the vehicles that can clear the intersection during different size gaps together, the total number of left-turning vehicles that can be discharged during the permissive phase can be estimated by the following equation:

$$\begin{aligned} S_{per}^{Poisson} &= \sum_{n=1}^{\infty} n * \text{number of } (nt \leq \text{gaps} \leq (n+1)t) \\ &= \sum_{n=1}^{\infty} n * Q_O * \left(e^{-Q_O nt/3{,}600} - e^{-Q_O (n+1)t/3{,}600}\right) \\ &= Q_O * \sum_{n=1}^{\infty} e^{-Q_O nt/3{,}600} \\ &= Q_O * \left(\frac{e^{-Q_O t/3{,}600}}{1 - e^{-Q_O t/3{,}600}}\right) \end{aligned} \quad (9)$$

where $S_{per}^{Poisson}$ is the saturation flow rate of permissive left-turn phase (veh/h) and $t$ is the critical gap (s), which is equal to the left-turning maneuver, $t_{LT}$.

However, it was found that in this study Equation 9 was valid only when the opposing volume was relatively low. When the opposing volume was high (close to capacity), the Poisson distribution assumption for the arrival pattern was not correct. For example, in the most extreme case, when the opposing volume ($Q_O$) is equal to the capacity of the opposing lanes, the gaps between the opposing vehicles are equal to the saturated headway (assumed to be 2 s). In this case, no left-turning vehicle could make a permissive left turn. To address that problem, in this study for the case in which the opposing volume

is greater than 400 vehicles per hour per lane (veh/h/lane), the following correction was made for estimating $S_{per}$:

$$S_{per} = \left(1 - \frac{Q_O}{N_O \times 1{,}800 \times \frac{T_B}{C}}\right) * S_{per}^{Poisson}$$

when opposing volume ≥ 400 veh/h/lane (10)

where $S_{per}^{Poisson}$ is the saturation flow rate estimated using Equation 9. Thus,

$$N_O \times 1{,}800 \times \frac{T_B}{C} = \text{capacity of opposing lanes}$$

The basic idea for this correction is to build a bridge between the most extreme case (all headway is 2 s) and the case that the Poisson arrival pattern assumption could be held. In this study it is assumed that when the opposing volume is less than 400 veh/h/lane, the arrival of opposing vehicles still follows a Poisson distribution. On the basis of Equation 10, the saturation flow rate during the permissive left-turn phase ($S_{per}$) can be estimated by the following equation:

$$S_{per} = \begin{cases} S_{per}^{Poisson} & \text{when opposing volume} < 400 \text{ veh/h/lane} \\ \left(1 - \dfrac{Q_O}{N_O \times 1{,}800 \times \dfrac{T_B}{C}}\right) * S_{per}^{Poisson} & \text{when opposing volume} \geq 400 \text{ veh/h/lane} \end{cases} \quad (11)$$

### Model Validation

To validate the proposed model, its output was compared with the data collected from the field. In addition, results were compared with the estimated results from a traditional method for estimating the left-turn capacity during the permissive phase, specifically the left-turn equivalency (ELT) method proposed by Roess (*15*). ELT is defined as the "through vehicle equivalent" for one left-turning vehicle. Conceptually, it is the number of through vehicles displaced in the traffic stream by one left-turning vehicle. In the ELT method let $S_T$ be the saturation flow rate of a through lane, then the saturation flow rate during the permissive left-turn movement can be calculated as $S_{per}^{ELT} = S_T / E_{LT}$. The value of the ELT factor is given in Table 18.1 of Roess (*15*).

The saturation flow rate during permissive left-turn movement cannot be directly observed in the field. The reason is that for intersections with low left-turn volume, not all available gaps will be used to discharge the left-turn vehicles. Thus, in the field, we can observe only the number of vehicles making permissive left turns. Therefore, in this study, instead of comparing saturation flow rates, researchers compared the total number of left-turn vehicles making turns during the permissive phase, which can be estimated as follows:

Step 1. Estimate the left-turn capacity during the permissive left-turn phase by both methods.

The left-turn capacity during the permissive phase can be estimated using the following equation

$$C_{per} = \frac{S_{per} \times T_E}{C} = \frac{S_{per} \times (T_B - T_D)}{C} \quad (12)$$

where

$C_{per}$ = estimated permissive left-turn capacity using the proposed method,
$S_{per}$ = estimated permissive left-turn saturation flow using the proposed method, and
$T_E$ = estimated effective green time for permissive left-turn movement using the proposed method.

The left-turn capacity during the permissive phase based on the ELT method can be estimated using the following equation:

$$C_{ELT} = \frac{S_{ELT} \times T_E}{C} = \frac{S_{ELT} \times (T_B - T_D)}{C} \quad (13)$$

where $C_{ELT}$ is the estimated permissive left-turn capacity using ELT method and $S_{ELT}$ is the estimated permissive left-turn saturation flow using ELT method.

Step 2. Estimate the number of vehicles making permissive left turns.

To estimate the total number of vehicles making turns during the permissive phase, the permissive left-turn capacity needs to be compared with the left-turn demand during the permissive phase. For the intersection approaches in the leading left-turn direction, the queue of left-turning vehicles will be cleared during the protected phase. Thus, the demand for permissive left-turn movement includes only the left-turning vehicles arriving during the permissive left-turn phase, that is, $T_B$. For intersection approaches in the lagging left-turn direction, because the permissive phase begins before the protected left-turn phase, there will be left-turn vehicles queued during the cross-movement phase, that is, $T_C$, and waiting to be discharged at the beginning of the permissive phase. Thus, the demand for permissive left-turn movement includes the left-turn vehicles arriving during the cross-movement phase, $T_C$, and the permissive left-turn phase, $T_B$. On the basis of the discussion above, the demand for permissive left-turn movement can be calculated using Equation 14.

$$D = \begin{cases} Q_L * \dfrac{T_B}{(T_A + T_B + T_C)} & \text{for leading LT phasing} \\ Q_L * \dfrac{(T_B + T_C)}{(T_A + T_B + T_C)} & \text{for lagging LT phasing} \end{cases} \quad (14)$$

where $D$ is the estimated demand for permissive left-turn movement.

Therefore, the total number of left-turn vehicles that can clear an intersection during the permissive phase per hour can be calculated using Equations 15 and 16.

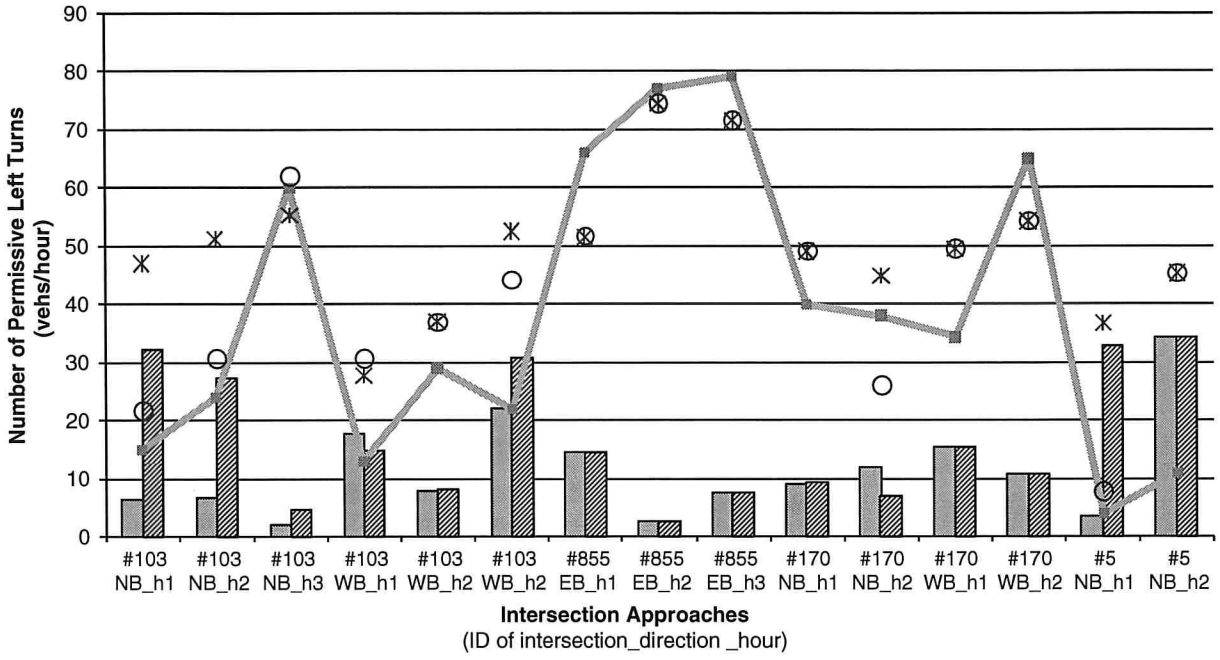
$$V_{per} = \min(C_{per}, D) \quad (15)$$

where $V_{per}$ is the estimated permissive left-turn volume using the proposed method.

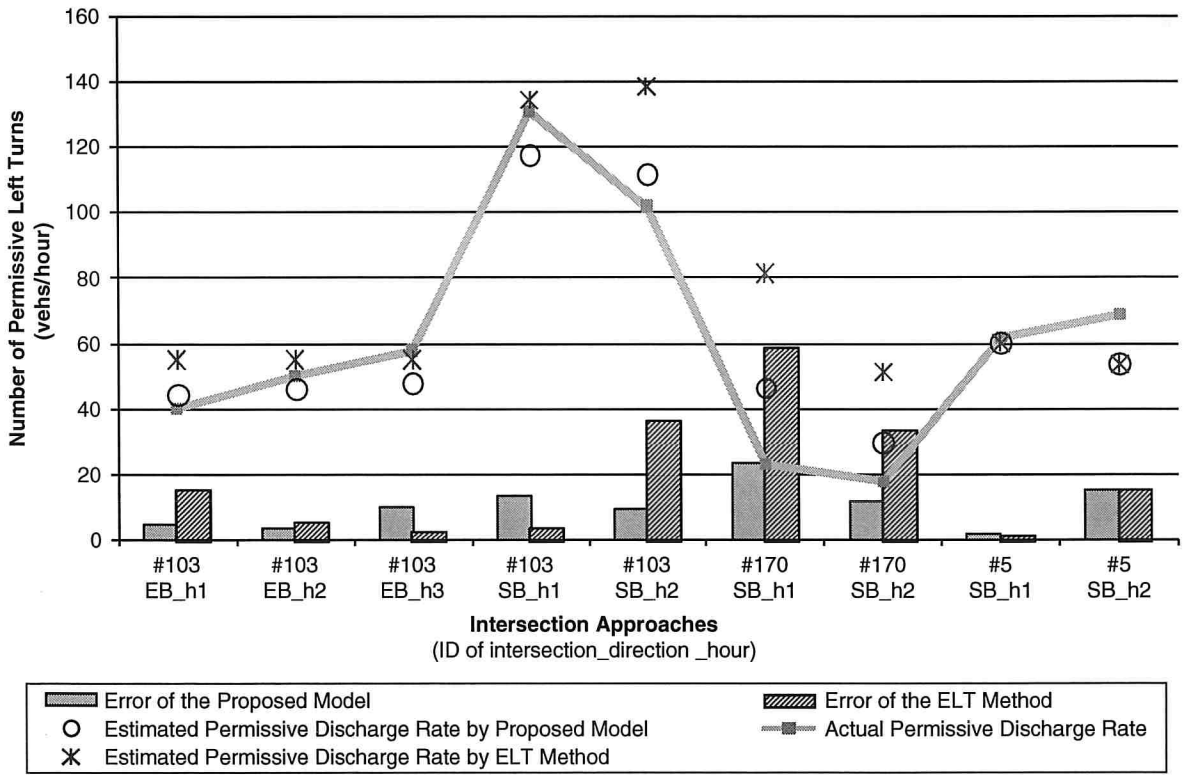
$$V_{ELT} = \min(C_{ELT}, D) \quad (16)$$

where $V_{ELT}$ is the estimated permissive left-turn volume using the ELT method.

The comparison of the permissive left-turn volumes for leading and lagging PPLT approaches is shown in Figures 4*a* and 4*b*, respectively. From Figure 4, it can be seen that the proposed method is much more accurate than the ELT method in estimating the permissive left-turn volume at some study intersection approaches,




(a)




(b)

FIGURE 4 Comparison between actual and estimated permissive left-turn volume: (*a*) leading left-turn intersection approaches and (*b*) lagging left-turn intersection approaches.

including Intersection 103 northbound (NB), Intersection 5 NB, Intersection 103 southbound (SB), and Intersection 170 SB. For other intersection approaches, the performance of the proposed method and the ELT method are very close. Overall, these results found that (*a*) for the leading left-turn approaches, the average error of the proposed model was about 11 vehicles per hour (veh/h) whereas the average error of the ELT method was about 17 veh/h and (*b*) for the lagging left-turn approaches, the average error of the proposed model was about 10 veh/h whereas the average error of the ELT method was about 19 veh/h. In addition, paired *t*-tests were conducted to compare the estimation errors of these two methods. Results of the *t*-tests give further proof that the proposed model can provide more accurate and reliable estimates of the permissive left-turn volume ($V_{per}$) than the ELT method (for leading left-turn approaches, *p*-value = .041; for lagging left-turn approaches, *p*-value = .067). These results validate the proposed methods for estimating the permissive left-turn saturation flow rate ($S_{per}$) because the estimated permissive left-turn volume ($V_{per}$) is a function of $S_{per}$.

With the $S_{per}$ estimated, the total left-turn delay reduction can be calculated using Equation 1 given the signal-timing plans. The procedure of estimating the left-turn delay reduction caused by the PPLT signal control mode included the following steps:

1. On the basis of the signal-timing plan and the traffic volume conditions of the study intersection approach, determine which queue diagram, that is, Figure 2*b*, Figure 2*c*, Figure 3*b*, or Figure 3*c*, should be used to analyze the delay of the study approach.
2. Determine the value of $T_D$, the portion of $T_B$ used to discharge the initial opposing through queue, using Equation 2.
3. Determine the value of $S_{per}$, the permissive left-turn saturation flow rate, using Equation 11.
4. Substitute these two variables along with other variables into Equation 1 to calculate the total left-turn delay reduction of the PPLT signal control mode.

## Cost Analysis

### Methodology

The cost of the PPLT mode over the PO mode is the increased safety risks caused by allowing vehicles to make left turns during the permissive phase. This study uses the concept of potential conflict gap (PCG) to quantify the level of safety risks at the study sites (*13*). The idea of the PCG method is based on the fact that most of the safety risk related to the permissive left-turn phase comes from the misjudgment of gaps in the opposing traffic, especially when the length of a gap is close to the actual left-turn maneuver time ($t_{LT}$). This is reasonable because if the opposing gap is very small, drivers will recognize it easily and yield. If the opposing gap is very big, the left-turning vehicles can clear the intersection with very low safety risks. Only when the opposing gap is close to the actual left-turn maneuver time, will it be most difficult for left-turning drivers to accurately estimate the gap and make the correct decision. Thus, this PCG is a very good indicator of safety risks. On the basis of this analysis, the PCG is defined as the opposing gap that is within a certain range of the actual left-turn maneuver time ($t_{LT}$). The number of PCGs during the permissive left-turn phase ($T_B$) will be estimated and serve as the measure of the safety cost of the PPLT mode over the PO mode.

Zhang et al. proposed a method to estimate the range of the PCG as follows (*12*):

$$\text{potential conflict gap} \in [t_l, t_u] \tag{17}$$

$$t_l = t_{LT} - \delta \tag{18}$$

$$t_u = t_{LT} - \delta \tag{19}$$

where

$t_{LT}$ = left-turning maneuver time, which could be estimated on the basis of the intersection's geometric conditions, average car length, and average acceleration rate;
$t_l$ = lower bound of PCGs, which could result in potential LT conflicts;
$t_u$ = upper bound of PCGs, which could result in potential LT conflicts; and
$\delta$ = reaction time of left-turning drivers, which is assumed to be 1.5 s.

Because it was assumed that the arrival of the opposing traffic follows a Poisson distribution, the probability that a gap is within the range of $[t_l, t_u]$ can be estimated as

$$P(t_l \le \text{gap} \le t_u) = e^{-Q_O t_l} - e^{-Q_O t_u} \tag{20}$$

where $P(t_l \le \text{gap} \le t_u)$ is the probability that a gap is within the range of $[t_l, t_u]$. Then, the number of potential conflicts, which are the events that the left-turning vehicles will take PCGs to make permissive left-turns, was estimated as

$$N_C = V_{per} * P(t_l \le \text{gap} \le t_u) = V_{per} * \left(e^{-Q_O t_l/3{,}600} - e^{-Q_O t_u/3{,}600}\right) \tag{21}$$

where

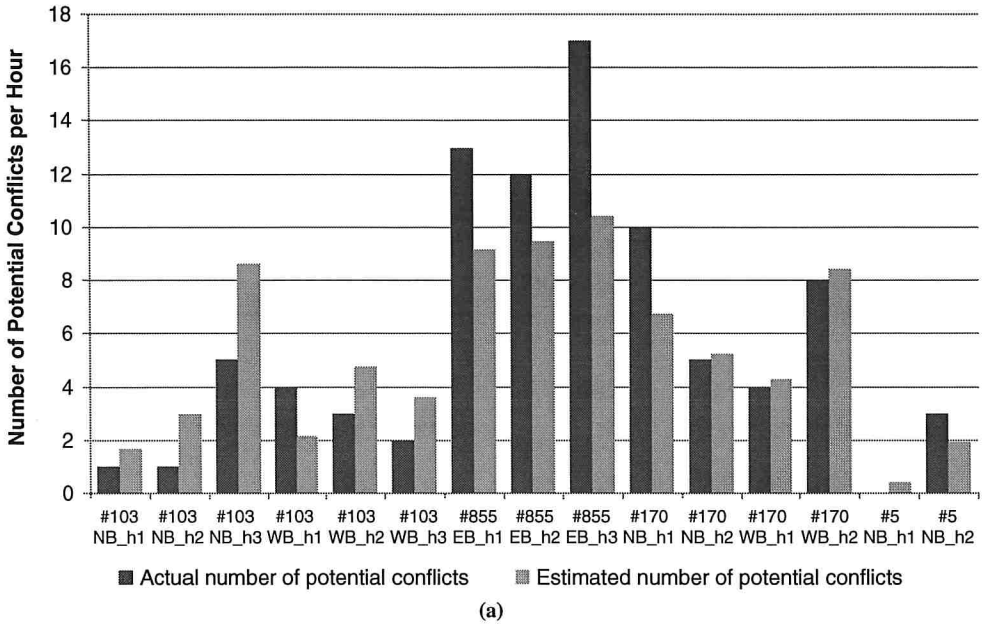
$N_C$ = number of potential conflicts;
$V_{per}$ = permissive left-turn volume in veh/h, which can be estimated by Equation 15;
$t_l$ = lower bound of PCG in seconds; and
$t_u$ = upper bound of PCG in seconds.

### Model Validation

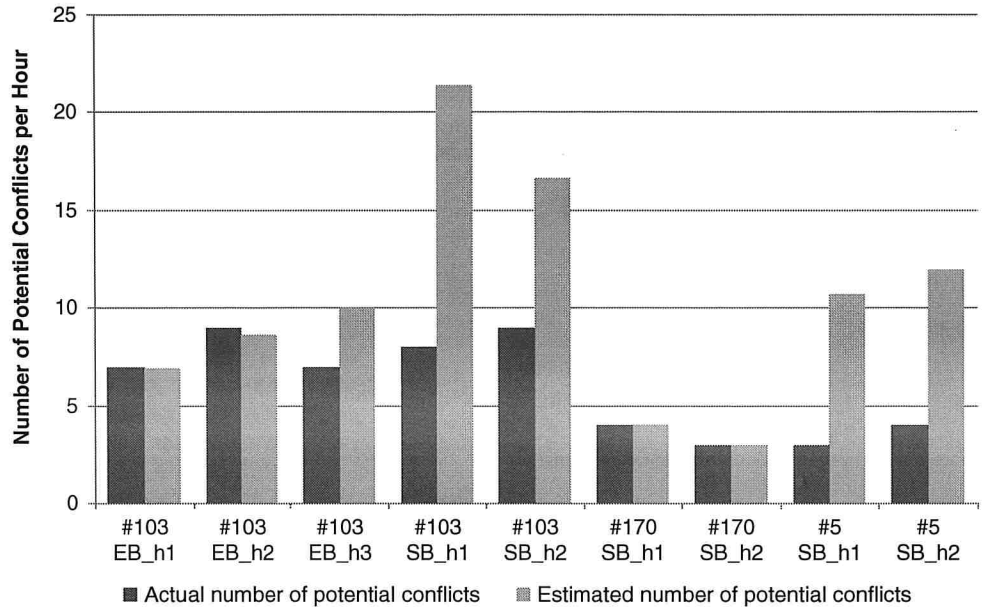
Model results were validated by comparing the estimated number of potential conflicts per hour at the study intersection approaches with the potential conflicts from data collected in the field. As shown in Figure 5, the predicted number of potential conflicts matched the observed number of potential conflicts quite well. The estimated results do not perfectly match the observed field data at some locations. The reason is that the different driving behaviors at the different locations affected the accuracy of the model results. Intersections that had a larger presence of aggressive drivers resulted in an increase in observed potential conflicts, and vice versa.

## Benefit and Cost Analysis

On the basis of the models developed in the previous two sections, the decision to select PPLT or PO can be made by conducting a benefit and cost analysis. First, the delay reduction of using the




(a)




(b)

FIGURE 5 Comparison of actual and estimated number of potential conflicts for (*a*) leading left-turn intersection approaches and (*b*) lagging left-turn intersection approaches.

PPLT mode needs to be estimated on the basis of the intersection traffic, signal timing, and geometric conditions using Equation 1. At the same time, the safety cost of the PPLT control mode, that is, the number of potential conflicts during the permissive phase, will be estimated according to Equation 21. Then, both the delay reduction and safety cost will be converted to a dollar value for comparison. A delay cost of $5.54 per vehicle per hour based on Agent was used in this study (*9*). Thus, the dollar value of the hourly delay reduction under the PPLT signal control mode can be estimated using the following equation:

$$\mathrm{DH} = \frac{\Delta D}{3{,}600} \times \left(\frac{3{,}600}{C}\right) = \frac{\Delta D}{C} \tag{22}$$

where DH is the hourly delay reduction of using the PPLT mode in vehicle hours and $\Delta D$ is the delay reduction of using the PPLT mode per cycle in vehicle seconds.

To determine the dollar value of potential conflicts, a web-based survey was conducted targeted at traffic engineers in different districts in Texas from June 1 to June 18, 2007. On the basis of the results of this survey, a cost of $4 per potential conflict was derived (*16*).

## Case Study

To demonstrate the application of the proposed models, a case study was conducted at the westbound (WB) approach of the 51st at Airport

intersection in Austin, Texas. At this intersection, the signal phasing is lead-lag PPLT and the WB is the leading left-turn approach. Traffic data were collected at this study site during 2 h of the a.m. peak period. The traffic data collected during the first hour were used for this case study. During this hour the left-turn lane volume was 126 veh/h and the opposing volume was 215 vph. There is one left-turn lane and two opposing through lanes. For the signal timing, the cycle length was 150 s, which consisted of 21 s of protected left-turn phase ($T_A$), 59 s of permissive left-turn phase ($T_B$), and 70 s of cross-movement phase ($T_C$). It was found that in most cases there was no left-turn queue at the end of the permissive left-turn phase ($T_B$).

### *Estimate the Delay Reduction of Using PPLT Mode*

On the basis of the provided information, the delay reduction of using the PPLT mode at this location was calculated using the following steps.

Step 1. Determine the queuing diagram for left-turn delay reduction estimation.

On the basis of the signal-timing plan and the traffic volume conditions of this study's intersection approach, the queuing diagram in Figure 2c was used to estimate the delay reduction of using the PPLT mode at this intersection approach.

Step 2. Determine the value of $T_D$ and $T_E$ using Equation 2.

$$T_D = T_{opp} = \frac{T_A + T_C}{C} \times Q_O \times h_O = \frac{21+59}{150} \times \frac{215}{\left(\frac{3{,}600}{150}\right)} \times 2 = 10.87$$

$$T_E = T_B - T_D = 59 - 10.87 = 58.13$$

Step 3. Estimate the permissive saturation flow rate ($S_{per}$) using Equation 10.

The opposing volume was less than 400 veh/h/lane; therefore,

$$S_{per} = S_{per}^{Poisson} = Q_O * \left(\frac{e^{-Q_O t_u/3{,}600}}{1 - e^{-Q_O t_u/3{,}600}}\right) = 215 * \left(\frac{e^{-215*6.0668/3{,}600}}{1 - e^{-215*6.0668/3{,}600}}\right)$$

$$= 492.37 \text{ veh/h}$$

Step 4. Estimate the reduction of delay ($\Delta D$) per cycle using Equation 1.

$$\Delta D = \frac{\frac{1}{2}QL}{3{,}600} * \left[\frac{S_{pro}(T_B+T_C)^2}{S_{pro} - Q_L} - \frac{S_{pro}T_C^2}{S_{pro} - Q_L} - \frac{S_{per}T_D^2}{S_{per} - Q_L}\right]$$

$$= 218.15 \text{ veh-s}$$

Step 5. Calculate the hourly delay reduction in vehicle hours ($DH$) by Equation 22.

$$\text{DH} = \frac{\Delta D}{\text{cycle length}} = \frac{218.15}{150} = 1.45 \text{ veh-h}$$

Step 6. Convert the hourly delay reduction into a dollar value.

$$\text{benefit} = \text{DH} * 5.54 = 1.45 * 5.54 = \$8.06$$

### *Estimate the Safety Cost of Using PPLT Mode*

Step 7. Estimate the number of potential conflicts per hour using Equation 21.

$$N_C = V_{per} * \left(e^{-Q_O t_l/3{,}600} - e^{-Q_O t_u/3{,}600}\right) = 34.29$$

$$* \left(e^{-215*4.5668/3{,}600} - e^{-215*7.5668/3{,}600}\right) = 4.28$$

Step 8. Convert the hourly safety cost into dollar value.

$$4.28 * \$4 = \$17.12$$

### *Compare the Operational Benefit and Safety Cost of Using PPLT Mode*

At this location, the delay reduction of using the PPLT mode is worth \$8.06/h, and the value of the potential conflicts caused by the use of the PPLT mode is \$17.12/h. Because the benefit of the PPLT mode is less than its safety cost, the PPLT mode is not recommended for this intersection approach during this peak hour. In this case study, researchers use just 1-h results to demonstrate the use of the proposed models. On the basis of the traffic conditions in other time periods, users can calculate benefits and costs in any other hours; then the final decision for the selection of appropriate signal control mode at this intersection can be made on the basis of the overall benefit and cost analysis.

## CONCLUSIONS AND FUTURE WORK

In this study analytical models for estimating the delay reduction and safety risk associated with the use of the PPLT control mode were developed. Researchers evaluated the accuracy of the developed models by comparing the model results with actual field data. Evaluation results demonstrated that the proposed models could provide accurate and reliable estimates for the delay reduction and potential conflicts caused by the use of the PPLT control mode. The proposed methodology overcame many of the limitations experienced with the traditional approaches for developing criteria for selecting between PPLT and PO, such as the requirement of reliable and adequate historical accident data, the constant thresholds for the traffic volume, and the lack of the ability to quantify and combine the operational and safety effects of the PPLT signal control mode.

However, the proposed models still have some limitations. Researchers will continue their efforts to improve the developed models in the future in the following aspects. First, in the intersection delay estimation it is assumed that the left-turn vehicles arrive at an intersection at a constant arrival rate, which could not exactly represent real-world traffic conditions, in which the vehicle arrival pattern is affected by various factors, such as platooning from other adjacent signalized intersections and side friction from driveways. In the future a more accurate and complicated model will be developed to take these factors into account. Second, this study used the traffic conflicts

rate to quantify the safety benefits of the PO operation over the PPLT operation. In the future, historical intersection accident records need to be collected to further validate model results. Third, the data for validating the developed models were collected from the intersections in one city, Austin, Texas, which may not accurately reflect drivers' gap acceptance behavior in other locations. In the future, to further validate the accuracy of the developed models, more data will be collected in different types of cities, such as big metropolitan areas and small towns in rural areas. Furthermore, some transportation agencies are now using "Flashing Yellow Arrow" signal indication to solve the yellow trap problem associated with the PPLT operation. Thus, research is needed in the future to investigate the effectiveness of the "Flashing Yellow Arrow" PPLT indication. Finally, the environmental benefits, such as the effects on emissions, also need to be assessed in selecting between PO and PPLT signal control modes.

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*The Traffic Control Devices Committee peer-reviewed this paper.*

# Evaluation of Effectiveness of Converging Chevron Pavement Markings in Reducing Speed on Freeway Ramps

Michael Hunter, Saroch Boonsiripant, Angshuman Guin, Michael O. Rodgers, and David Jared

**Converging chevron pavement markings have recently seen rising interest in the United States as a means to reduce speeds at high-speed locations and improve safety performance. This paper reports on an investigation into the effectiveness of chevron markings in reducing vehicle speeds on two-lane freeway-to-freeway directional ramps in Atlanta, Georgia. The evaluation is based on a statistical comparison at preselected sites of speeds before and after the installation of the chevron markings. The analysis focuses on the effect of converging chevrons over the range of speed percentiles and on the mean speed. The analysis indicates that chevrons have a minimal effect on vehicle speeds, with drivers adjusting back to their previous speeds as they acclimate to the treatment. The effect of the chevrons' treatments on speed tended to be most pronounced immediately after the chevron implementation. However, by the 9th month after implementation the magnitude of the effect dropped to under 1 to 2 mph for the mean speed and most vehicle speed percentiles. Although this result does not necessarily imply that the chevron treatment is not a meaningful safety treatment, any safety benefits are not likely to result from a general decrease in speeds.**

Speed is a prime factor in highway safety. In 2007, total traffic fatalities amounted to 41,059 in the United States, of which 13,040 were speeding-related fatalities, accounting for 31% of total fatalities. Of the total number of speeding-related fatalities in the United States in 2007, 1,204 (9%) occurred on roads with posted speed limits above 55 mph (*1*). As such, measures for discouraging speeding, especially at high-speed locations, have been of major interest in the traffic engineering community. First proposed nearly a decade ago in Japan, converging chevron pavement markings (abbreviated as "chevron markings" hereafter) have recently seen rising interest in the United States. The first documented U.S. test of chevron markings was undertaken in Milwaukee County, Wisconsin, in 1999. This experiment concluded that "converging chevron pavement markings appear to reduce speeds but more research is needed" (*2*).

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This paper reports on an investigation into the effectiveness of chevron markings in reducing vehicle speeds on two-lane freeway-to-freeway directional ramps in a desire to improve safety performance. The evaluation is based on a statistical comparison of speeds before and after the installation of chevron markings at preselected sites in the Atlanta, Georgia, area. The analysis focuses on the effect of converging chevrons over the range of speed percentiles and on the mean speed through the use of diverse sampling and control ramps. Although no such study can be definitive, this evaluation study may be considered part of the systematic nationwide effort to assess the effectiveness of chevron markings in reducing speeds in the United States.

## BACKGROUND

### Introduction

Pavement markings are used as a means to guide or inform the road user of local and federal regulations and the presence of potentially hazardous locations (*3*). The advantage of the pavement markings over other traffic control devices (e.g., signs and traffic signals) is that pavement markings allow motorists to focus on the road while the information is being communicated. The *Manual on Uniform Traffic Control Devices* (MUTCD) lists four basic types of pavement markings (*4*):

- Longitudinal lines, for example, centerlines, edge lines, and lane lines;
- Transverse lines, for example, stop line, yield line, and crosswalk markings;
- Arrows, words, and symbol markings; and
- Special markings, for example, raised pavement markers and two-way left-turn lane markings.

The two most common types of pavement markings used to influence drivers' speed choice are longitudinal and traverse markings (*3, 4*). Several studies indicated a speed reduction for narrower lane widths (*5–10*), and longitudinal markings provide one method for lane width reduction. More commonly, transverse pavement markings are used as a speed reduction countermeasure. Generally transverse pavement markings are installed ahead of horizontal curve sections, intersection approaches, work zones, and freeway off-ramps to warn road users of potentially hazardous locations (*3, 11*). These markings are normally a series of transverse lines in a bar or chevron pattern across the travel

lane. Spacing between these lines may decrease, as with the converging chevrons, as the driver approaches the potentially hazardous location. The decreasing spacing intends to create a perception of acceleration to drivers, which in turn encourages them to reduce their speed (*3*). Short bars placed only on the edge of the travel lane are called peripheral transverse lines (*12, 13*).

### Previous Studies

Several earlier studies evaluated the effectiveness of pavement markings as a speed reduction countermeasure (*2, 3, 12–16*). Some of the earlier studies most relevant to this evaluation are summarized below.

The first converging chevron pavement markings in the United States were installed in 2003 at an exit ramp location in Milwaukee, Wisconsin. In a study of their effectiveness Drakopoulos and Vergou found mean and 85th percentile speed reductions of 15 and 17 mph at the downstream detector of the chevron treatment (*2*). Crash data were also presented in the report showing that the number of crashes decreased at the test and the comparison ramps. However, the authors stated that the crash data were available only for a short time frame and thus a statistical crash analysis could not be conducted.

A study in Kentucky evaluated the effectiveness of several treatments (i.e., warning sign, post delineators, transverse lines, flashers) used on three selected rural sites (*3*). The speed data revealed mixed results. One site indicated no significant overall speed reduction but considerable speed variance reduction for most treatments, including transverse markings. The other two sites showed significant speed reductions for all treatments. For all sites, the 85th percentile and greater speed were affected more significantly by the treatments than lower speed values, including the mean speeds.

A comprehensive study by Katz evaluated the effectiveness of peripheral transverse lines on speed reduction on a freeway exit ramp and two rural arterials (*13*). This study also showed a mixed effect of the treatment. The freeway exit ramp (located in New York) showed the most significant speed reduction at approximately 4 mph in mean speed and 5 mph in the 85th percentile speed; the rural arterials (located in Mississippi and Texas) were affected only slightly by the treatment. The author explained that several factors influenced the magnitude of the treatment effect, including driver familiarity with the road, degree of curvature, and visibility of the pavement markings.

The most recent study by Anthony et al. evaluated a freeway connector ramp in El Paso, Texas (*17*). Vehicle speeds were measured at three locations along the curve during three time periods to determine short-term and long-term effects of the treatment. Detailed analysis was conducted by vehicle classification, light condition, and location along the curve. It was found that speed reduction varied on the basis of vehicle class and curve location. The maximum effect measured was on heavy trucks, with a reduction of 4 mph at the upstream location.

### Speed as Safety Surrogate Measure

A primary purpose of the pavement marking treatment is to warn road users of a potentially hazardous location and encourage them to reduce their speeds. Because higher speed at impact increases the severity of the collision, the reduction of speed at the hazardous location is expected to improve road safety (*15, 18, 19*). Therefore, the magnitude of speed reduction is often used in the literature as a safety surrogate measure of the effectiveness of pavement marking treatments.

Another measure that can be used as a safety surrogate measure is speed variance. Several studies indicate that high speed variance associates with high crash risk (*20–24*).

In summary, three forms of speed-related measures are commonly used to evaluate the effectiveness of transverse pavement marking treatments (*15*):

- Reduction of mean speed,
- Reduction of operating or 85th percentile speed, and
- Reduction of speed variance.

## EXPERIMENTAL DESIGN

### Study Sites

The effectiveness of converging chevron pavement markings has been evaluated at two interchange locations in Atlanta, Georgia:

- Site A: Interstate 75/85 interchange and
- Site B: Interstate 75/285 interchange.

The study sites are shown in Figure 1. The driving factor influencing the selection of these ramps was the desire to test the chevron treatment at freeway-to-freeway ramp locations at which the ramp geometry requires a significant decrease in vehicle speeds. The minimum ramp curve radii on Site A and Site B are approximately 150 ft and 500 ft, respectively, and the desired direct effect of the chevron implementation is a reduction of vehicle speeds before this controlling ramp geometry, potentially resulting in an improved safety performance of the ramps.

To monitor potential underlying changes in traffic operations at the interchange, for each site two ramps were selected—a treatment ramp, that is, the ramp containing the chevron installation; and a control ramp, that is, a ramp with no treatment applied.

At Site A, the treatment ramp is the two-lane interchange ramp from I-75 southbound to I-85 northbound. This ramp serves approximately 18,000 vehicles per day per lane (vpdpl) during the week and 14,000 vpdpl during the weekend. There are approximately 2% heavy vehicles. Shown in Figure 2*a* are the data collection stations installed at this interchange. Data collection stations are placed immediately upstream (S011) and downstream (S012) of the chevron marking location. The distance between the two stations is approximately ¼ mi. The average speeds at the upstream and downstream stations before chevron marking installation are 51 and 31 mph, respectively. There are two sets of rumble strips upstream of the chevron treatment on Site S011 that were in place at least 2 years before the study and remained unchanged throughout the study period. Also seen in Figure 2*a* are the Site A control ramp data collection stations, S033 and S032. These collect the traffic data for the control ramp from I-85 southbound to I-75 northbound. These data collection stations were chosen to encompass the location of a chevron installation had this been a treatment ramp.

At Site B the two-lane interchange ramp from I-285 eastbound to I-75 northbound was selected as the treatment ramp, and the interchange ramp from I-75 southbound to I-285 westbound serves as the control ramp. The treatment ramp serves approximately 18,600 vpdpl during weekdays and 16,700 vpdpl during the weekend with 6% truck traffic. The treatment ramp is composed of two data collection

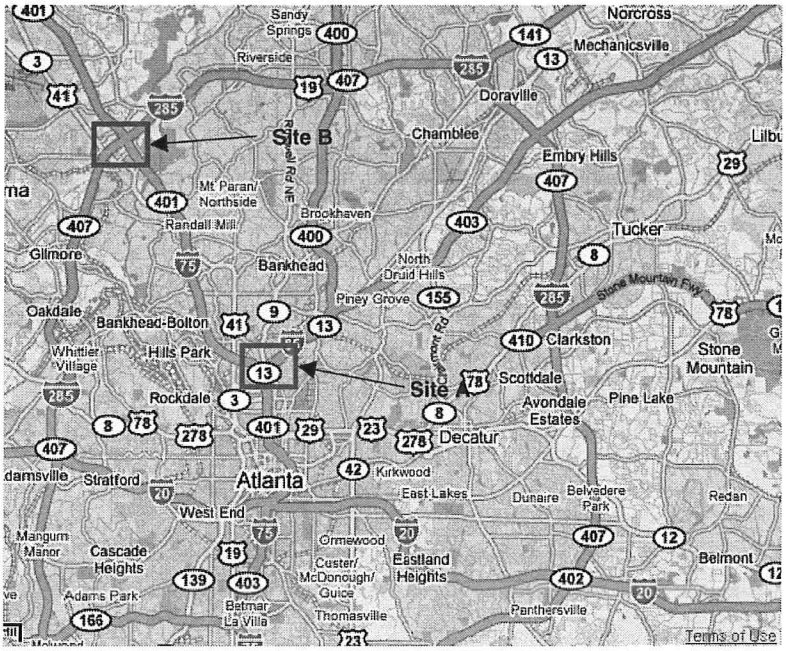



FIGURE 1 Converging chevron treatment study sites (*25*).

stations. Station S021 is located upstream of the chevron marking location, and Station S022 is located immediately downstream of the marking location. The distance between the stations is approximately ¼ mi. Average speeds at the upstream and downstream stations before chevron marking installation are 60 and 45 mph, respectively. In addition, data collection equipment was placed at the upstream (S041) and downstream (S042) stations of the Site B control ramp. As with Site A the control ramp data collection points were selected to be representative of the locations that would be selected had this been a treatment ramp. The locations of data collection stations at Site B are depicted in Figure 2*b*.

## Chevron Design

The treatment ramp Site A contains 23 sets of chevrons on each lane, with the full chevron treatment covering a distance of 815 ft; the treatment ramp Site B contains 22 sets of chevrons covering a distance of 750 ft. Each chevron set contains 10 individual chevrons and is approximately 20 ft long. The spacing between the two most upstream groups of chevrons is 21 ft. This spacing is reduced for each subsequent set of chevrons, with the two most downstream chevron sets at approximately a 10-ft spacing. A photo of the chevron treatment at the Downstream Detector Station S022 of Site B is seen in Figure 3.

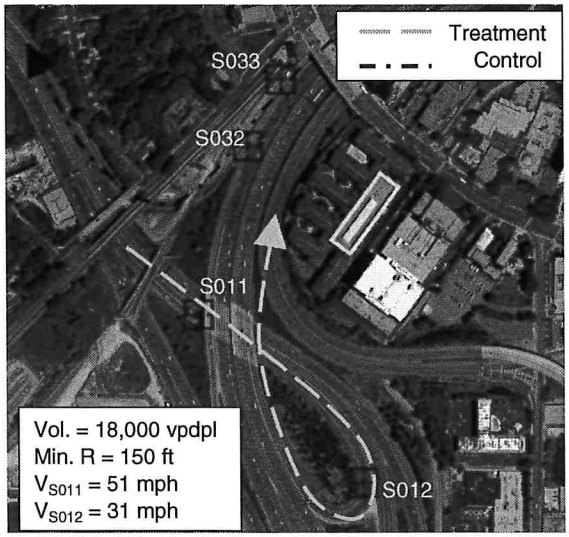
## Data Descriptions

Each detection station consists of a dual loop with a piezoelectric detection device between two inductance loops. The piezoelectric device is intended to allow for an accurate measurement of the distance between axles and increase the accuracy of the classification of the vehicles. Streaming per vehicle record (PVR) data were recorded in the field and periodically downloaded for analysis. The PVR data for each vehicle contained parameters including time stamp, lane number, vehicle class, vehicle speed, vehicle length, time headway, and distance between axles. The data were collected between March 2008 and February 2009. This study compared speed distributions during six time periods, including

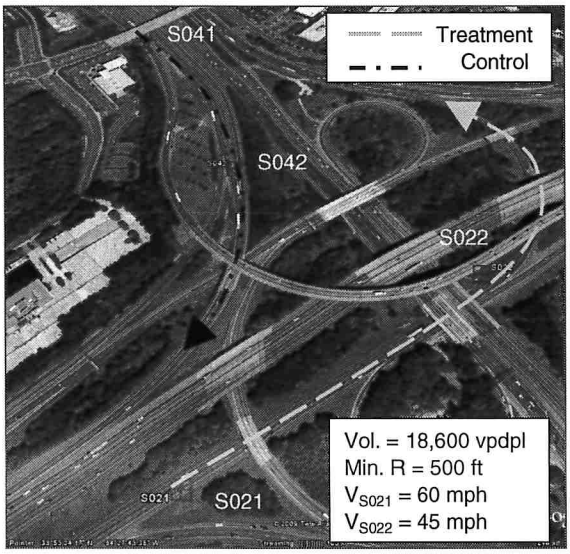
- Before the treatment (5 weeks),
- One week after the treatment (1 week),
- One month after the treatment (1 week),
- Three months after the treatment (2 weeks),
- Six months after the treatment (1 week), and
- Nine months after the treatment (1 week).

## Data Quality Analysis

As part of the equipment installation the speed detections were calibrated by the equipment vendor. Speeds were also spot-checked by the research team using a laser gun. On the basis of the vendor calibrations and checked vehicle speeds, it was deemed that the equipment generally provided a high level of accuracy. However, occasionally speeds were recorded by the detection device significantly different from that observed. This typically was a result of a vehicle changing lanes while crossing the detector pairs. Speed calibration was conducted while the traffic was moving at typical (uncongested) speeds. Detectors were not calibrated for congested scenarios, and they become potentially less accurate as speeds drop below 10 mph. In addition, the equipment used for data collection natively supported the collection of binned data, allowing for a data consistency check by comparing traffic counts from the PVR and binned data. Under high-demand conditions the ability of the detection device to stream PVR data could be strained, resulting in some missed vehicles. Finally, once the initial data collection was completed the devices were not recalibrated during the study period. Because the critical measure was the relative change in speeds, not




(a)




(b)

FIGURE 2 Data collection stations with travel directions for test (yellow line) and control (red line) sites: (*a*) Site A, Interstate 75/85 interchange, and (*b*) Site B, Interstate 75/285 interchange (*26*).

the absolute speed measure, a determination was made to leave the detector setting once the project was under way.

## DATA ANALYSIS

The initial analysis considers detectors located downstream of each ramp treatment, that is, Site A, Detector Site S021; and Site B, Detector Site S022. These detector locations should reflect any change in speed resulting from the chevron treatments before vehicles enter the controlling ramp geometry. For each of these detector locations, speed distributions for each data collection time period were recorded during the data collection process. These data are examined in regard to direct and cumulative speed distributions and

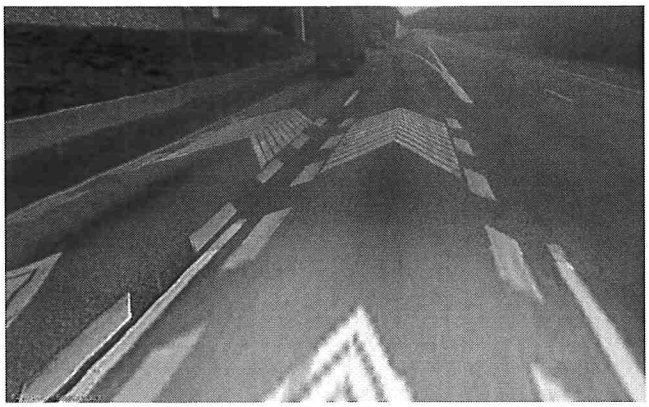


FIGURE 3 Chevron treatment before Detector Station S022 of Site B (*26*).

the way these distributions change across time periods. Lead-vehicle (i.e., free flow) and Monte Carlo–based analyses incorporating the upstream detectors and control ramps are also conducted.

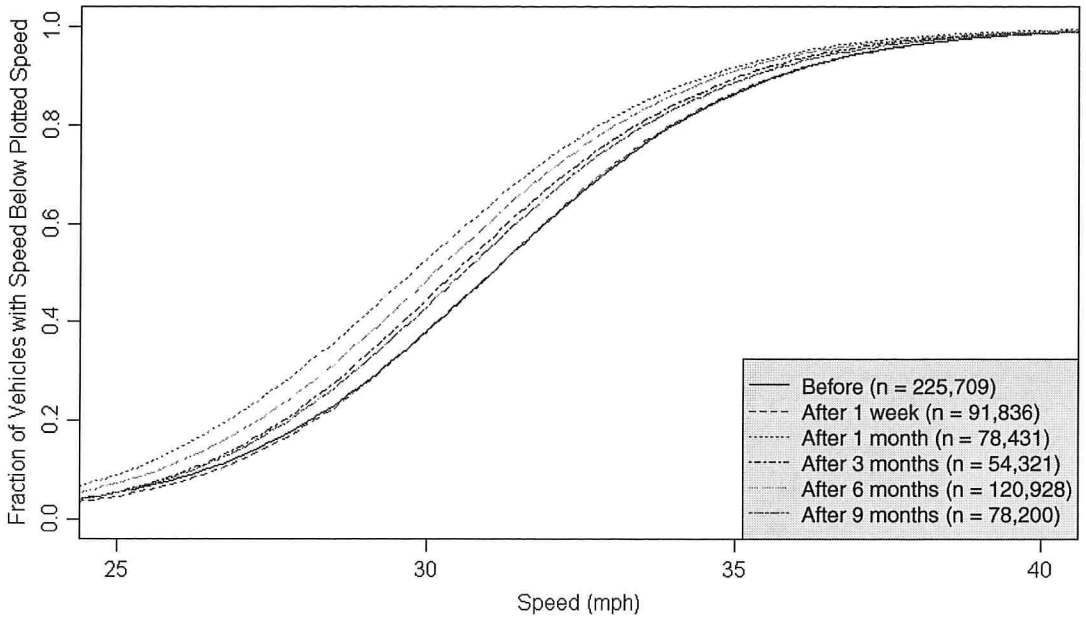
### Speed Cumulative Distribution Functions

Figures 4*a* and 4*b* illustrate the cumulative distribution function (CDF) of the PVR speed data for Lane 1 (the rightmost lane) of Detection Sites S012 and S022, respectively.

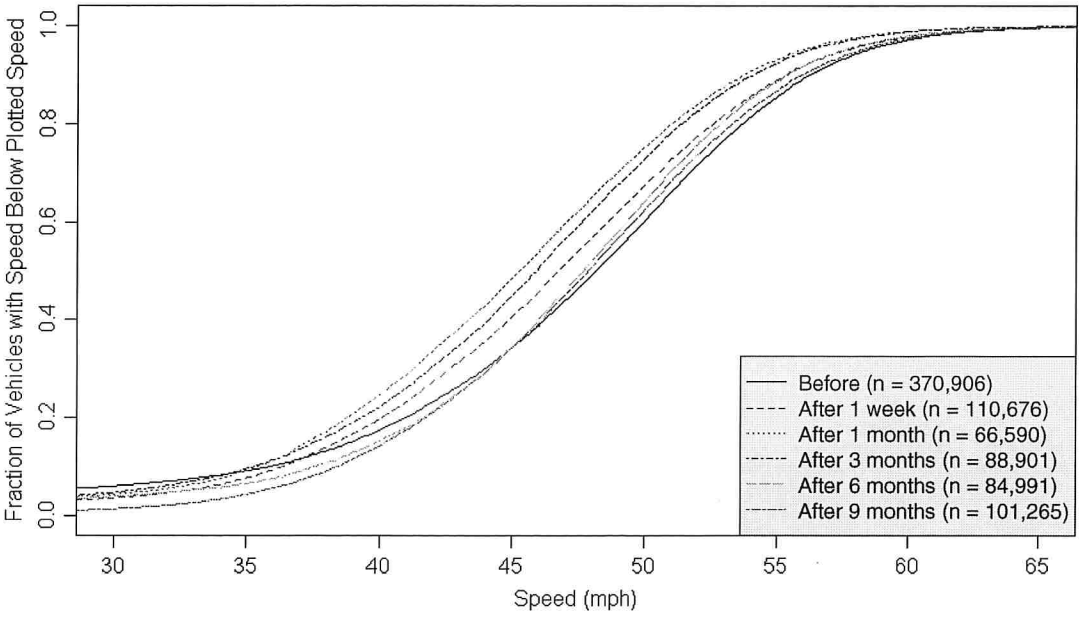
Each line corresponds to speed data during different time periods. The "n" in the legend represents the number of data points available. Figure 4*a* depicts the distribution shifts at Site S012, at which there is shifting of ramp speeds to lower (slower) levels for 1, 3, 6, and 9 months after treatment, with the largest change observed after 1 month. By the 9-month data collection period the largest observed shift at any percentile is on the order of 0.5 to 1 mph. Data from Site S022, illustrated in Figure 4*b*, show the same general trend with slower speeds observed after the chevron treatment installation; however, in this case speeds have generally returned to the pretreatment levels by the 9-month period. The only significant differences in the 9-month data at this site are seen in the lower percentile, in which fewer low-speed vehicles are observed in the before-treatment data. Similar results were seen in the Lane 2 (the leftmost lane) data for Detector Site Locations S012 and S022. Only Lane 2 of Site S022 demonstrated any consistent speed reduction at the 9-month period, generally on the order of 1 to 2 mph.

### Analysis of Speed Difference

To further illustrate the differences in the speed distributions before and after treatment, the speeds at each percentile of the distribution are compared with their pretreatment values. Figure 5*a* illustrates these speed differences for Site S012, Lane 1, and Figure 5*b* shows results for Site S022, Lane 1. In these plots a negative value indicates the speed reduction for the after-treatment time period compared with the before-treatment values, and a positive value indicates a speed increase. For example, in Figure 5*a*, at the 40th percentile (i.e., 0.4) the speed difference between the before-treatment and the 1-month data is approximately 1.5 mph, reducing to approximately 0.5 mph by the 9-month period, as was earlier noted from observation




(a)



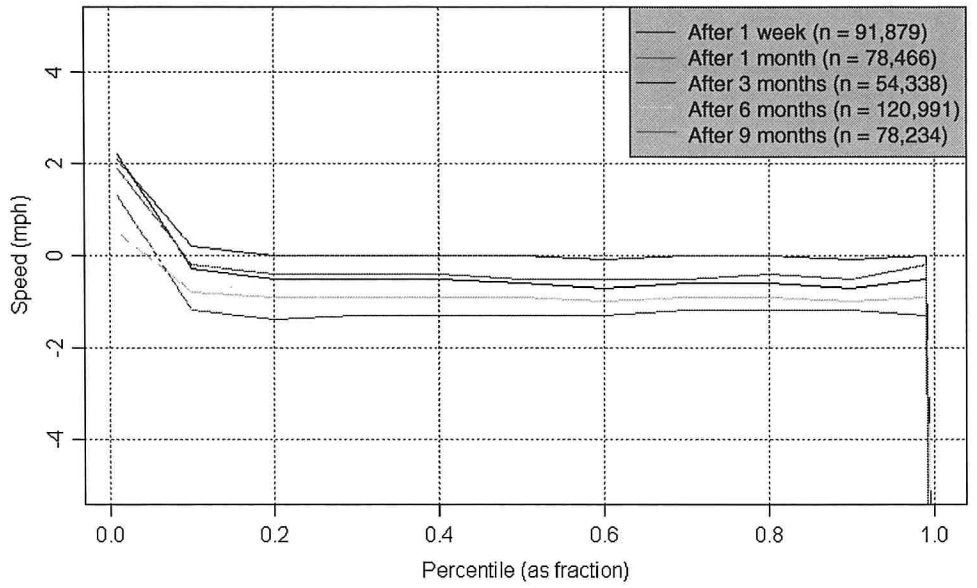

(b)

FIGURE 4 CDF plot with PVR data (*n* denotes number of data points in the parentheses corresponding to each period): (*a*) Site S012, Lane 1, and (*b*) Site S022, Lane 1.

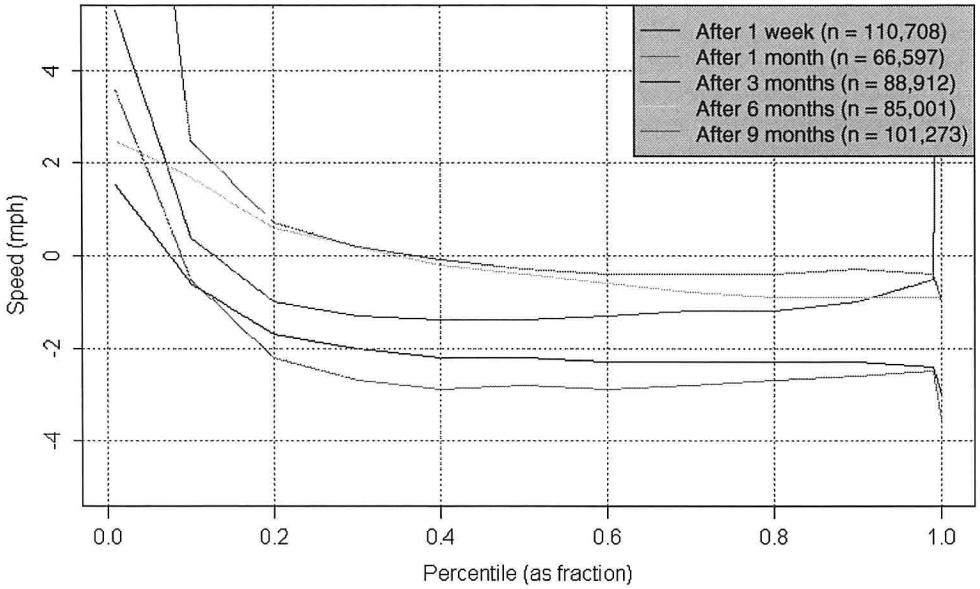
of the CDF plot. Further, Figure 5*a* shows that speed reductions are observed over a broad percentile range; from approximately the 5th to past the 99th percentile, the speed reductions at the 9-month period are on the order of 0.5 mph. Although there are some clear shifts in the data, the size of the deviations appears to be very small over most of the distributional range. Figure 5*b* illustrates the same effects, relative minimal effects by the 9-month period over the range of the speed distributions, with the lower percentile speeds seen to increase by the 9-month period. Findings for the Lane 2 speeds on the Site S012 and S022 detectors are also similar.

## Lead Vehicle Analysis

As stated earlier an expected effect of the chevron treatment is a reduction in driver speeds. However, if a vehicle is closely following another vehicle, as is typical in congested conditions, it is not known whether the subject vehicle reduces speeds as a result of roadway conditions (e.g., the ramp geometry, chevrons, signage, etc.) or in response to the actions of a vehicle in front of the subject vehicle. Thus far the analysis for each time period has used all available data. For the lead vehicle analysis the vehicles are separated into leading




(a)




(b)

FIGURE 5 Spread between CDF plots with PVR data for all vehicles: (*a*) Site S012, Lane 1, and (*b*) Site S022, Lane 1.

(i.e., platoon leaders) and following vehicles. Any vehicle with a headway of 5 s or more (measured at the upstream detector) is considered a leading vehicle for this analysis. The general assumption of a lead vehicle, or platoon leader, is that a lead vehicle is free to travel at its desired speed. Thus, lead vehicles should more clearly indicate any effect of the chevron treatment. The CDF speed differences for the leading vehicles are plotted in Figure 6 for Site S012, Lane 1. Although the sample size ($n$) is almost halved, the magnitude and direction of the shifts of the percentile speed differences for the leading vehicles are similar to the spread overall (Figure 5). That is, the lead-vehicle analysis results in nearly the same speed distribution, and speed difference, as the all-vehicle analysis. This same result is also seen in the lead-vehicle analysis results for the remaining Site S012 and Site S022 results.

## Monte Carlo Analysis

Monte Carlo analysis uses random sampling to allow for an unbiased statistical evaluation of the mean difference between data sets with different population parameters. For example, in the first Monte Carlo analysis of this effort a vehicle speed is randomly selected from the before data and a vehicle speed is randomly selected from the after data, for a detector location. This creates a before–after speed-data pair.

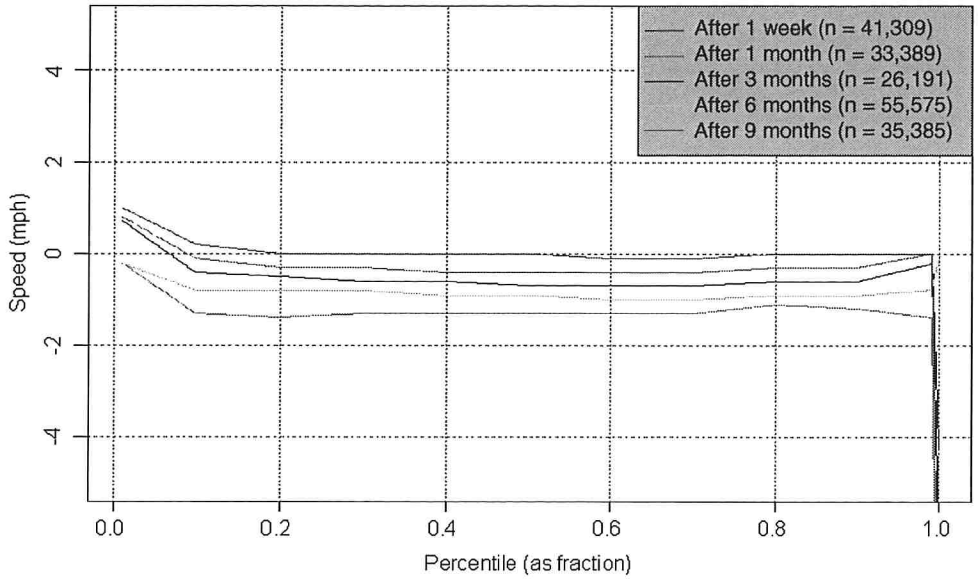



FIGURE 6 Spread between CDF plots with PVR data at Site S012 Lane 1 for leading vehicles.

The difference between the individual speeds of this data pair represents a single sample of the before–after speed difference. By drawing multiple samples the average of these differences is an unbiased estimate of the mean speed difference between the time periods. In this effort several different Monte Carlo–based analyses were performed:

1. Estimate of the difference in the mean speed at the downstream treatment detector locations across the before and after time periods,
2. Estimate of the difference in the change in speed between the upstream and downstream treatment detectors across the before and after periods,
3. Estimate of the difference in the mean speed at downstream control detector locations across the before and after time periods, and
4. Estimate of the difference in the change in speed between the upstream and downstream control ramp detectors across the before and after periods.

The first analysis was undertaken for the downstream treatment detectors, that is, Detectors S012 (Site A) and S022 (Site B). This analysis allows for a more formal estimate than that found in the earlier discussion of the speed difference observed as the vehicles enter the controlling ramp geometry. The second analysis seeks to determine whether there is a difference in the speed reduction through the chevron treatment area. This analysis explores the possibility that a background trend in increasing or decreasing traffic speeds has masked the treatment effect. For example, the earlier discussion implied a minimal effect on speed at the downstream detectors by the 9th month. It is possible that during this time period background speeds had increased, resulting in a higher mean speed of vehicles entering the chevron treatment zone. If the vehicle speeds exiting the treatment zone remained unchanged, then the speed reduction through the zone would have increased between the before and after time periods. By considering only the downstream detectors, this potential treatment effect could be overlooked. The third and fourth Monte Carlo analyses are similar to the first and second, except that they are conducted on the control ramp data. The control ramp findings may then be compared with the treatment ramp findings to determine whether the measured effects (or lack of) are a result of the chevron treatment or a background traffic trend. Table 1 summarizes results for the before treatment versus 9th-month comparisons. For brevity this discussion focuses on this 9-month data. For each analysis three iterations of the Monte Carlo analyses were performed. Each analysis iteration drew a sample of 2,000 data points (with replacement) from a population of roughly 100,000 vehicles in each period. Each vehicle speed was chosen randomly from each period's data set. Averages are computed across the three iterations using a pooled estimate. Only lead-vehicle speeds are used for the Monte Carlo analysis.

### *Estimate of Difference in Mean Speed at Downstream Treatment Detector Locations Across the Before-and-After Time Periods—Treatment and Control Results*

The first results discussed from the Monte Carlo analysis are the speed on the treatment ramps as the vehicles cross the downstream detectors, that is, enter the controlling ramp geometry. The earlier analysis suggested that the chevrons have a minimal effect on speeds in this location. The Monte Carlo analysis supports this assessment, with a mean measured speed decrease at Site A of 0.19 mph on the leftmost lane and 0.29 on the rightmost lane. Site B showed similar results with a mean speed increase of 0.44 and speed decrease of 0.48 on the leftmost and rightmost lanes, respectively. Of the Site A and B results only the 0.29 was found to be statistically significant. Also there is minimal difference in the before and 9-month confidence intervals, indicating that the chevrons had little effect on the speed variance. This observation agrees with visual observation of the CDF plots (not shown for brevity).

Site B control ramps do exhibit some increase in the speed of vehicles entering the controlling ramp geometry, with increases of 0.85 mph and 1.48 mph on the leftmost and rightmost lanes, respectively. Both of these differences were found to be statistically significant. Assuming similar behavior may have been observed on the treatment ramp, this would indicate a potential 0.5-mph to 2.0-mph speed reduction as a result of the chevrons (that is, subtracting the measured control ramp effect from the measured treatment ramp effect). The Site A control ramp exhibited a statistically significant 0.18-mph

**TABLE 1 Monte Carlo Results**

| Speed Measurement | Ramp Type | Detector | Before Treatment (95% CI) (mph) | Nine Months After Treatment (95% CI) (mph) | Mean Diff. Ninth Month—Before (95% CI) (mph) | Mean Diff. Significant at 95% Level |
|---|---|---|---|---|---|---|
| Downstream detector mean | Site A treatment | S012, left lane | 31.74 (31.55, 31.92) | 31.55 (31.34, 31.75) | −0.19 (−0.47, 0.09) | No |
| | | S012, right lane | 32.42 (32.25, 32.60) | 32.13 (31.97, 32.29) | −0.29 (−0.53, −0.06) | Yes |
| | Site A control | S032, left lane | 40.37 (40.13, 40.61) | 40.55 (40.31, 40.78) | 0.18 (−0.15, 0.51) | Yes |
| Reduction between detectors | Site A treatment | S011 to S012, left lane | −20.28 (−20.63, −19.92) | −21.91 (−22.28, −21.55) | −1.64 (−2.15, −1.13) | Yes |
| | | S011 to S012, right lane | −21.06 (−21.43, −20.70) | −22.31 (−22.65, −21.97) | −1.24 (−1.74, −0.74) | Yes |
| | Site A control | S033 to S032, left lane | −16.99 (−17.55, −16.44) | −17.16 (−17.75, −16.56) | −1.05 (−1.79, −0.32) | Yes |
| Downstream detector mean | Site B treatment | S022, left lane | 48.03 (47.65, 48.41) | 48.47 (48.16, 48.78) | 0.44 (−0.05, 0.93) | No |
| | | S022, right lane | 48.16 (47.76, 48.57) | 47.68 (47.37, 47.99) | −0.48 (−0.99, 0.03) | No |
| | Site B control | S042, left lane | 51.54 (51.20, 51.88) | 52.39 (52.08, 52.71) | 0.85 (0.39, 1.32) | Yes |
| | | S042, right lane | 52.59 (52.22, 52.97) | 54.07 (53.76, 54.39) | 1.48 (0.99, 1.96) | Yes |
| Reduction between detectors | Site B treatment | S021 to S022, left lane | −12.32 (−12.90, −11.75) | −13.63 (−14.09, −13.16) | −1.30 (−2.04, −0.56) | Yes |
| | | S021 to S022, right lane | −11.61 (−12.23, −10.99) | −14.16 (−14.63, −13.70) | −2.55 (−3.32, −1.77) | Yes |
| | Site B control | S041 to S042, left lane | −0.22 (−0.72, 0.28) | −3.22 (−3.66, −2.78) | −3.00 (−3.67, −2.33) | Yes |
| | | S041 to S042, right lane | −0.68 (−1.21, −0.15) | −3.04 (−3.52, −2.56) | −2.36 (−3.08, −1.64) | Yes |

NOTE: CI = confidence interval and Diff. = difference.

change in speed. Combined with the observed treatment ramp results, the overall effect of the chevrons remains under 0.5 mph.

### *Estimate of Difference in the Change in Speed Between Upstream and Downstream Treatment Detectors Across the Before-and-After Periods—Treatment and Control Ramps*

In estimating the difference in the change in speed between the upstream and downstream ramp, the speed difference between these locations is determined for each time period and then the difference in the speed reductions between analysis periods is determined. For the Site B treatment ramp, increases in speed reduction between detectors from the before to the 9th-month after of 1.31 and 2.54 are seen for the leftmost lane and rightmost lanes, respectively. That is, compared with the before period, speed reduction is 1.31 mph greater on the leftmost lane and 2.54 mph greater on the rightmost lane after 9 months of chevron treatment installation. In the Monte Carlo estimate of speeds entering the controlling ramp geometry on the treatment ramps little effect was seen between the before and 9-month data; thus, most of the observed increase in speed reduction between detectors is due to higher chevron zone entering speeds in the after period. Similar speed reductions are seen on the Site A ramps, at 1.63 mph and 1.25 mph on the leftmost and rightmost ramps, respectively.

In the final Monte Carlo analysis the speed reduction between detectors was observed for the control ramps. A review of Table 1 shows reductions similar to those on the treatment ramps, with Site B control ramp reductions even slightly higher than those on the treatments. Given that the control ramps also experience the increase in speed reduction between detectors, the effect on the treatment ramp cannot be associated with the treatment itself but instead is likely due to changes in background conditions.

### *Monte Carlo Summary*

When analyzing the speed entering the controlling ramp geometry, it is seen that the speed reduction is on the order of 0.5 to 2.0 mph, with much of this effect related to increased speed on the control ramp. This finding corresponds well with the earlier CDF analysis. In addition, there is minimal effect on the speed variance of vehicles entering the controlling ramp geometry. When considering the potential for speed reductions between the detectors, it would appear that the chevrons had little to no effect, with the control ramps experiencing trends similar to that of the treatment ramps. Taken together this would imply that the mean effect of the chevrons is limited to 0.5-mph to 2.0-mph reduction in speed as the vehicles enter the controlling ramp geometry.

A parallel analysis using only the heavy trucks in the vehicle mix revealed similar shifts. The differences between the results of the all-vehicle analysis and the heavy-truck analysis were typically less than 0.5 mph, and the variation trends were also similar. For brevity, details of this analysis are not included.

## SUMMARY OF FINDINGS

The speed data analysis indicates that the chevrons had only a modest effect on vehicle speeds. The observed changes in the speed distributions are reasonably consistent between the test sites, allowing

for confidence in the findings. The effect of the chevrons' treatments on speed tended to be most pronounced immediately after the chevron implementation, with the effect waning by the 9th month and the magnitude of the effect under 1 to 2 mph over most of the vehicle speed percentiles.

To account for sampling differences and potential distributional difference between the data collected during the different time periods, a Monte Carlo random sampling strategy of lead vehicles was adopted. An analysis of the speed entering the controlling ramp geometry shows that the average speed reduction is on the order of 0.5 to 2.0 mph by the 9th month, with much of this effect related to increased speeds on the control ramp. In addition, there is minimal effect on the speed variance of vehicles entering the controlling ramp geometry. When considering the potential for speed reductions between the detectors, it would appear that the chevrons had little to no effect, with the control ramps experiencing trends similar to that of the treatment ramps. Taken together this would imply that the mean effect of the chevrons is limited to a 0.5-mph to 2.0-mph reduction in mean speed as the vehicles enter the controlling ramp geometry.

In summary, the effect of the chevron treatment on speed tends to be minimal, with drivers adjusting back to their previous speeds as they acclimate to the treatment. However, this does not necessarily indicate that the chevrons are not a meaningful safety treatment but rather that any safety benefits are likely not due to a general decrease in speeds. For example, it is possible that the chevron treatment may help alert an inattentive driver, thus reducing the likelihood of an accident occurrence without having a significant effect on the overall population. In addition, an incident analysis for the subject ramps showed that a significant subset of the crashes occurred under wet or snowy conditions and it is possible that the chevrons function differently under these adverse conditions. Data collection constraints precluded examining this possibility. Before any final judgment on the effectiveness of the chevron treatment on safety, it is recommended that a direct accident study (as opposed to using a surrogate such as speed) be conducted after sufficient accident data have been gathered. Also the minimal speed reduction is based on two, two-lane freeway ramp sites in one geographic area, and additional research is needed. Although this study did not consider the effects of sight distance or vertical curvature, as there is no notable sight distance issue or vertical curvature present through the chevron area of both sites, if a crash analysis reveals significant safety benefits, future research will investigate the effect of such factors in that context.

## ACKNOWLEDGMENTS

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*The Traffic Control Devices Committee peer-reviewed this paper.*

# Guidance on Design and Application of Rumble Strips

Darren J. Torbic, Jessica M. Hutton, Courtney D. Bokenkroger, Karin M. Bauer, Eric T. Donnell, Craig Lyon, and Bhagwant Persaud

**Many transportation agencies use shoulder rumble strips to address the problem of single-vehicle run-off-the-road crashes by alerting inattentive or drowsy motorists that their vehicles have drifted out of the travel lane. The application of rumble strips has expanded to include the installation of centerline rumble strips along the centerlines of undivided highways to reduce head-on and opposite-direction sideswipe crashes. Installing rumble strips along either the shoulder or centerline without considering the effect on other highway users (i.e., bicyclists and motorcyclists) may lead to unintended consequences. This research addresses a number of safety issues: (*a*) the safety effectiveness of shoulder rumble strips on different roadway types, (*b*) the safety effectiveness of shoulder rumble strip placement relative to the edgeline, (*c*) the safety effectiveness of centerline rumble strips on different roadway types, and (*d*) the safety effectiveness of centerline rumble strips along horizontal curves and tangents. The safety evaluations considered all severity levels (total crashes) and fatal and injury crashes. Statistical models for predicting noise levels in the passenger compartment of a vehicle for use in designing rumble strip patterns were also developed. The results of this research were combined with results from previous research to address important policy issues for transportation agencies to consider in the design and application of shoulder and centerline rumble strips.**

Many transportation agencies use shoulder rumble strips (SRS) to address the problem of single-vehicle run-off-the-road (SVROR) crashes by alerting inattentive or drowsy motorists that their vehicles have drifted out of the travel lane. The application of rumble strips has expanded to include installation of centerline rumble strips (CLRS) along the centerlines of undivided highways to reduce head-on and opposite-direction sideswipe crashes. Installing rumble strips along either the shoulder or centerline without considering their effect on other highway users (i.e., bicyclists and motorcyclists), however, may lead to unintended consequences.

A significant amount of research has been conducted on rumble strips. However, a number of issues related to the design and application of shoulder and centerline rumble strips remain unresolved. This paper presents results from research conducted to address several of these issues.

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## OBJECTIVE AND SCOPE

The objective of this paper was to develop further guidance for the design and application of shoulder and centerline rumble strips as an effective motor vehicle crash reduction measure. The research addressed the following issues:

- Safety effectiveness of SRS on different roadway types,
- Safety effectiveness of SRS placement relative to the edgeline,
- Safety effectiveness of CLRS on different roadway types,
- Safety effectiveness of CLRS along varying roadway geometry, and
- Optimum dimensions of rumble strips necessary for effective vehicular warning with least potential for adverse effects on bicyclists.

This research focused on milled rumble strips, so unless specified otherwise assume that the text is referring to milled rumble strips. For more detailed information concerning this research, refer to Torbic et al. (*1*).

## RESEARCH METHODOLOGY OF SAFETY EVALUATIONS

Safety evaluations were performed to address the first four objectives above. The primary data collection effort involved a videolog review of treatment and nontreatment sites. The review served

1. To confirm the presence or absence of rumble strips at candidate treatment and nontreatment sites,
2. To confirm site characteristic data obtained from roadway inventory databases,
3. To record roadside hazard ratings (RHR) [RHR system developed by Zegeer et al. to characterize the accident potential for roadside designs on two-lane roads (*2*)] to characterize the roadsides of sites,
4. To confirm the construction history of sites (i.e., to determine installation dates and appropriate years for the analyses),
5. To estimate offset distances of SRS relative to the edgeline, and
6. To record the locations of tangents and horizontal curves and note the presence or absence of curve warning signs at horizontal curves.

Two statistical approaches were used to evaluate whether installing rumble strips has an effect on crash frequencies:

- Before–after comparison using the empirical Bayes (EB) method and
- Cross-sectional analysis using a generalized linear model (GLM) approach.

The EB method is now a widely used method to evaluate the safety effectiveness of a countermeasure given a set of before–after sites and reference sites. The EB method, which adjusts for the effects of regression to the mean, is based on the comparison of observed crash frequencies in the after period with predicted crash frequencies in the after period, had the treatment not been implemented. The EB method used in this analysis is based on methods recommended by Hauer (*3*). Safety performance functions (SPFs) were developed for each crash type, roadway type, and state considered in the analyses and are documented in the final project report (*1*). A negative binomial model was used to estimate the regression coefficients by the maximum likelihood method.

A cross-sectional analysis was performed to determine the statistical association of rumble strip placement relative to the edgeline on the safety effectiveness of SRS. A GLM with a negative binomial distribution and a log link was used to model the yearly crash counts. A repeated measures correlation structure was included to account for the relationship in crashes at a given site across years. A compound symmetry covariance structure was used. General estimating equations were used to determine the final regression parameter estimates. Full details on the calibrated models are in the final project report (*1*).

## SAFETY EFFECTIVENESS OF SHOULDER RUMBLE STRIPS

The objectives of the safety evaluation of SRS were to

- Quantify the safety effectiveness of milled SRS on specific roadway types, including freeways, multilane divided highways, multilane undivided highways, and two-lane roads in rural and urban areas, and
- Quantify the safety effectiveness of milled SRS placed in varying locations relative to the edgeline.

The safety evaluation investigated the change in crash frequency for total (TOT) crashes, fatal and injury (FI) crashes, SVROR crashes, and SVROR FI crashes. A SVROR crash was defined as any single-vehicle crash involving a vehicle leaving the traveled way that was not intersection- or ramp-related.

### Safety Effectiveness of SRS on Different Roadway Types

Data for this evaluation were collected in Minnesota (MN), Missouri (MO), and Pennsylvania (PA). Although the evaluation was to quantify the safety effectiveness of milled SRS for eight roadway types, insufficient data prohibited the evaluation of several roadway types. Therefore, the safety effectiveness of milled SRS was analyzed for the following four roadway types:

- Urban freeways,
- Rural freeways,
- Rural multilane divided highways, and
- Rural two-lane roads.

Table 1 presents the number of before–after sites included in the analyses, the mile years (i.e., miles × years in the analysis period) of data before and after installation of the rumble strips at the treatment sites, and results from the EB analyses by individual state and combined across states. Because of space considerations, Table 1 does not present information concerning the SPFs developed for use in the EB calculations. The discussion related to the results presented in Table 1 focuses on SVROR (and SVROR FI) crashes for the following reasons. First, SVROR crashes are the types of crashes that will most likely be directly affected by SRS. Second, no strong argument can be made to support why SRS would affect crashes other than SVROR crashes. Third, many of the analysis results for TOT crashes were counterintuitive. Therefore, the analysis results for TOT and FI crashes should be viewed with caution. In summary, the conclusions of the safety evaluations for SRS are based on results for SVROR and SVROR FI crashes. The conclusions are also based on results obtained from analyses of combined data sets, rather than results for an individual state. In general, the results across analysis approaches were most consistent when based on the combined data.

As presented in Table 1, the average safety effects (and their standard error, SE) of installing milled SRS on rural freeways were estimated to be

- 10% reduction in SVROR crashes (SE = 5) and
- 17% reduction in SVROR FI crashes (SE = 7).

The average safety effects (and their standard error, SE) of installing milled SRS on rural two-lane roads were estimated to be

- 16% reduction in SVROR crashes (SE = 8) and
- 36% reduction in SVROR FI crashes (SE = 10).

For rural multilane highways, although the result for SVROR crashes showed a statistically significant result, the result was counterintuitive. This appears to be an anomaly in the data for this roadway type. A number of factors working in combination could explain this result, such as (*a*) the unreliability related to property-damage-only crash records, (*b*) design features of this roadway type, and (*c*) driver population along this roadway type. Also the sample size for this roadway type is relatively small, making it difficult to control for possible confounding factors. Because the result of the analysis of SVROR crashes on rural multilane divided highways did not pass a face validity test (i.e., results were in the expected direction and the magnitude appeared reasonable), this result was viewed as an anomaly in the data and was not considered credible. For urban freeways, results for SVROR and SVROR FI crashes were not statistically significant.

Results from this research for rural freeways and two-lane roads were combined with results from Griffith (*4*) and Patel et al. (*5*). Results from Griffith are believed to be the most reliable results from previous safety evaluations of SRS for rural freeways (*4*), and the study by Patel et al. was the only previous safety evaluation of SRS on rural two-lane roads (*5*). Results were combined in a manner consistent with procedures for combining study results for incorporation in the forthcoming *Highway Safety Manual* (HSM) (*6*). Table 2 presents the safety effectiveness estimates of SRS in the form of accident modification factors (AMFs). These estimates are viewed

**TABLE 1 Safety Effectiveness of SRS on Four Crash Types**

| | | | | | Crash Type | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | TOT | | | FI | | | SVROR | | | SVROR FI | | |
| Roadway Type | State | Number of Sites | Mile Years Before | Mile Years After | Change in Crash Frequency from Before to After Rumble Strip Installation (%) Estimate[a] | SE[b] | Test Statistic[c] | Change in Crash Frequency from Before to After Rumble Strip Installation (%) Estimate[a] | SE[b] | Test Statistic[c] | Change in Crash Frequency from Before to After Rumble Strip Installation (%) Estimate[a] | SE[b] | Test Statistic[c] | Change in Crash Frequency from Before to After Rumble Strip Installation (%) Estimate[a] | SE[b] | Test Statistic[c] |
| Urban freeways | Pennsylvania | 53 | 182 | 130 | −1.4 | 5.7 | 0.24 | −16.0 | 7.2 | 2.21 | −5.8 | 7.3 | 0.79 | −7.4 | 9.9 | 0.75 |
| Rural freeways | Combined | 47 | 393 | 213 | 7.0 | 3.9 | 1.80 | −6.9 | 5.9 | 1.17 | −9.7 | 5.2 | 1.86 | −17.1 | 7.3 | 2.35 |
| | Missouri | 29 | 313 | 156 | 7.9 | 4.1 | 1.91 | −5.8 | 6.4 | 0.91 | −7.9 | 5.7 | 1.38 | −15.6 | 8.2 | 1.90 |
| | Pennsylvania | 18 | 80 | 57 | 0.3 | 11.8 | 0.03 | −12.6 | 14.6 | 0.86 | −17.7 | 12.3 | 1.44 | −23.2 | 15.7 | 1.48 |
| Rural multilane divided highways | Combined | 25 | 243 | 116 | 18.1 | 7.8 | 2.32 | −10.2 | 10.2 | 0.99 | 40.0 | 12.4 | 3.23 | −2.6 | 13.5 | 0.20 |
| | Minnesota | 6 | 100 | 60 | 10.2 | 14.7 | 0.70 | −22.2 | 19.6 | 1.13 | 38.4 | 26.6 | 1.44 | −10.3 | 28.6 | 0.36 |
| | Missouri | 14 | 109 | 51 | 22.0 | 9.5 | 2.32 | −5.2 | 12.3 | 0.43 | 44.8 | 14.8 | 3.03 | 0.2 | 15.8 | 0.01 |
| | Pennsylvania | 5 | 33 | 5 | −13.3 | 35.6 | 0.37 | −40.1 | 42.5 | 0.94 | −25.5 | 37.4 | 0.68 | −19.9 | 56.9 | 0.35 |
| Rural two-lane roads | Combined | 53 | 558 | 387 | 5.9 | 5.7 | 1.03 | −8.0 | 8.0 | 0.99 | −16.2 | 8.1 | 2.01 | −36.4 | 9.7 | 3.75 |
| | Minnesota | 28 | 478 | 286 | 14.4 | 8.0 | 1.80 | 5.1 | 12.7 | 0.41 | 10.7 | 17.1 | 0.63 | −32.4 | 17.6 | 1.84 |
| | Missouri | 5 | 64 | 31 | 40.5 | 18.0 | 2.25 | −19.2 | 21.8 | 0.88 | 16.9 | 21.8 | 0.78 | −44.6 | 23.2 | 1.93 |
| | Pennsylvania | 20 | 136 | 70 | −24.4 | 8.6 | 2.83 | −18.0 | 11.6 | 1.55 | −43.6 | 9.1 | 4.77 | −36.7 | 13.3 | 2.75 |

[a]A negative percent change indicates a decrease in crash frequency, while a positive effect indicates an increase in crash frequency.
[b]SE = standard error of estimate.
[c]Test statistic = abs(estimate/SE); not significant at 90% confidence level if <1.7; significant at 90% confidence level if ≥1.7; significant at 95% confidence level if ≥2. A row is shaded in gray whenever a percentage change from the analysis shows a statistically significant rumble strip effect at the 95% or 90% confidence level.

TABLE 2 AMFs for Shoulder Rumble Strips Recommended for Inclusion in the HSM

| Treatment | Roadway Type | Accident Type and Severity | AMF[a] | SE[b] |
|---|---|---|---|---|
| Shoulder rumble strips[c] | Rural freeways | SVROR | 0.89 | 0.1 |
| | | SVROR FI | 0.84 | 0.1 |
| Shoulder rumble strips[d] | Rural two-lane roads | SVROR | 0.85 | 0.1 |
| | | SVROR FI | 0.71 | 0.1 |

[a]AMF = accident modification factor.
[b]SE = standard error of estimate.
[c]AMF and SE based on combined results for rolled SRS from Griffith (*4*) and for milled SRS from this research.
[d]AMF and SE based on combined results from Patel et al. (*5*) and this research.

as the most comprehensive and reliable estimates for the safety effectiveness of SRS to date. The safety effectiveness estimates for rural freeways in Table 2 suggest application to SRS in general, rather than specifically to milled or rolled rumble strips.

To provide further guidance on the safety effectiveness of SRS on other roadways types, it was assessed from the literature review that the best available estimates of the safety effectiveness of SRS for other roadway types are as follows:

- Rolled SRS on urban freeways are expected to reduce SVROR crashes by 18% and SVROR FI crashes by 13% (*4*) and
- Milled SRS on rural multilane divided highways are expected to reduce SVROR crashes by 22% and SVROR FI crashes by 51% (*7*).

These estimates are not considered as reliable as the estimates presented in Table 2 on the basis of the analysis approach, sample sizes, or both but are considered the best to date for the respective roadway types.

## Impact of Rumble Strip Placement on Safety Effectiveness of SRS

The effect of shoulder rumble strip offset from the edgeline was evaluated using a cross-sectional analysis approach. The analysis focused on SVROR FI crashes. Three levels of analyses were performed by dividing the offset distances into categorical levels and by considering the recovery area in the analysis. The offset distance was measured relative to the inside edge of the edgeline separating the outside travel lane from the shoulder; so, for example, if the rumble strip was installed on the edgeline, the offset distance would be 0 in. (0 mm), whereas if the rumble strip was installed adjacent to the edgeline but not on the edgeline, the offset distance was estimated to be 4 in. (102 mm) because most standard edgelines are 4 in. (102 mm) in width. Recovery area was calculated as the difference between shoulder width and rumble strip offset.

The categorical levels considered for the three analyses were as follows:

- Level 1—edgeline versus nonedgeline rumble strip effect compared with no rumble strips:
  - Edgeline rumble strips [offsets of 0 to 8 in. (0 to 203 mm)],
  - Nonedgeline rumble strips [offsets of 9+ in. (229+ mm)], and
  - No rumble strips.
- Level 2—effect of offset distance at three levels compared with no rumble strips:
  - 0 to 8 in. (0 to 203 mm),
  - 9 to 20 in. (229 to 508 mm),
  - 21+ in. (533+ mm), and
  - No rumble strips.
- Level 3—effect of combination of offset and recovery area compared with no rumble strips:
  - Edgeline rumble strip and 4+ ft (1.2+ m) recovery area,
  - Edgeline rumble strip and 0–4 ft (0–1.2 m) recovery area,
  - Nonedgeline rumble strip and 4+ ft (1.2+ m) recovery area,
  - Nonedgeline rumble strip and 0–4 ft (0–1.2 m) recovery area,
  - No rumble strip and 4+ ft (1.2+ m) shoulder width, and
  - No rumble strip and 0–4 ft (0–1.2 m) shoulder width.

Results of just the first-level analysis are presented here.

Table 3 presents results of the analyses designed to determine the effect that rumble strip placement relative to the edgeline has on the safety effectiveness of SRS. Table 3 shows the estimated percent difference in crash frequency as a function of rumble strip offset distance, its 95% confidence limits (CL) and *p*-value, and the Type 3 *p*-value. Generally, the two *p*-values are in agreement with each other; however, when the two disagree, the Type 3 *p*-value is used to draw final conclusions (*8*).

Table 3 shows two sets of analyses for rural two-lane roads. In one analysis, all the data from MN, MO, and PA were included, resulting in a combined set of 257 sites. In the other, the data for rural two-lane roads were analyzed without the 53 nontreatment MN sites because these sites accounted for almost half the MN rural two-lane roads and could unduly influence results. This exclusion influences only the MN analysis results and the combined multistate analysis results for rural two-lane roads, presented in the last two rows of Table 3.

The most relevant results from this analysis are for rural freeways (combined) and for rural two-lane roads (combined). For rural freeways, analysis of the combined data shows statistically significant results at the 90% confidence level on the basis of the Type 3 *p*-value. Estimates for the percent change in crash frequency attributed to offset of a given distance were as follows:

- 29% reduction in SVROR FI crashes for edgeline rumble strips and
- 9% reduction in SVROR FI crashes for nonedgeline rumble strips.

For rural freeways, these results provide evidence that offset distance affects the safety effectiveness of SRS.

Analysis of the combined state data for rural two-lane roads shows a slightly different picture. Focusing first on the analysis of combined data using all available 257 sites, results indicate a statistically significant reduction in crashes for edgeline and nonedgeline SRS. On average, a 33% reduction in SVROR FI crashes was estimated for edgeline SRS, and a 46% reduction was estimated for nonedgeline SRS. Given the relative magnitude of the difference between these two estimates, there is not much difference compared with the difference between the two estimates for rural freeways. The minimal difference between the two estimates for edgeline versus nonedgeline SRS is clearer in the analysis based on the 204 combined sites that excludes the 53 MN nontreatment sites. These results indicated a 39% reduction

**TABLE 3 Safety Effectiveness of Rumble Strip Placement on SVROR FI Crashes**

| Roadway Type | State | Number of Sites | Number of Site Years | RS Placement | Percent Difference in Crash Frequency with Rumble Strips Present (%)[a] Estimate | Lower 95% CL | Upper 95% CL | *p*-Value | Type 3 *p*-Value |
|---|---|---|---|---|---|---|---|---|---|
| Urban freeways | Pennsylvania | 138 | 999 | Nonedgeline | 1.7 | −15.7 | 22.6 | .86 | .87 |
| Rural freeways | Combined | 122 | 776 | Edgeline | −28.8 | −51 | 3.5 | .08 | .07[b] |
| | | | | Nonedgeline | −8.9 | −23.7 | 8.6 | .30 | |
| | Missouri | 47 | 351 | Edgeline | −24.8 | −49.5 | 12 | .16 | .22 |
| | | | | Nonedgeline | −2.9 | −17.5 | 14.3 | .73 | |
| | Pennsylvania[c] | 75 | 425 | Edgeline | | | | | |
| | | | | Nonedgeline | | | | | |
| Rural multilane divided highways (nonfreeways) | Combined | 104 | 788 | Edgeline | −25.1 | −46.5 | 4.9 | .09 | .16 |
| | | | | Nonedgeline | 10.3 | −17.3 | 47.1 | .50 | |
| | Minnesota | 60 | 424 | Edgeline | −29.6 | −50.6 | 0.5 | .05 | .05[d] |
| | | | | Nonedgeline | 23.4 | −6.3 | 62.5 | .14 | |
| | Missouri | 27 | 239 | Edgeline | −41.7 | −63.5 | −6.8 | .02 | .13 |
| | | | | Nonedgeline | 28.3 | −17.8 | 100.2 | .27 | |
| | Pennsylvania | 17 | 125 | Edgeline | 31 | 11.6 | 53.9 | <.001 | .09[b] |
| | | | | Nonedgeline | −47.7 | −73.5 | 3.2 | .06 | |
| Rural two-lane roads | Combined | 257 | 2,124 | Edgeline | −33.3 | −53.1 | −5.2 | .02 | .03[d] |
| | | | | Nonedgeline | −46.5 | −67.1 | −13.1 | .01 | |
| | Minnesota | 109 | 726 | Edgeline | 8.3 | −25.8 | 58 | .68 | .82 |
| | | | | Nonedgeline | −8.2 | −43.9 | 50.4 | .74 | |
| | Missouri[c] | 38 | 366 | Edgeline | | | | | |
| | | | | Nonedgeline | | | | | |
| | Pennsylvania[c] | 110 | 1,032 | Edgeline | | | | | |
| | | | | Nonedgeline | | | | | |
| Rural two-lane roads[e] | Combined | 204 | 1,872 | Edgeline | −39.2 | −62.5 | −1.5 | .04 | .05[d] |
| | | | | Nonedgeline | −41.9 | −69.4 | 10.0 | .10 | |
| | Minnesota | 56 | 474 | Edgeline | −42.9 | −70.5 | 10.5 | .10 | .27 |
| | | | | Nonedgeline | 10.5 | −42.3 | 111.8 | .76 | |

NOTE: CL = confidence limit.
[a]Percent change is relative to no rumble strips. A negative percent change indicates a decrease in crash frequency, while a positive effect indicates an increase in crash frequency.
[b]Significant at 90% confidence level.
[c]GLM algorithm did not converge.
[d]Significant at 95% confidence level.
[e]Excludes 53 Minnesota nontreatment cross-sectional sites.

in SVROR FI crashes for edgeline rumble strips compared with a 42% reduction for nonedgeline rumble strips. For practical purposes, a 3% difference between the two estimates indicated that offset distance does not affect the safety effectiveness of SRS along rural two-lane roads.

Possible explanations for the difference in results between rural freeways and rural two-lane roads may include the following:

- Different driving populations having different attitudes and behaviors,
- Differences in driving behavior while driving along a multilane divided roadway in which opposing traffic is separated by a median and adjacent traffic is traveling in the same direction versus driving along a two-lane road without any type of physical separation between opposing vehicles traveling in the opposite direction,
- Differences in design standards (e.g., related to horizontal and vertical alignments), and
- Extreme differences in roadside characteristics (e.g., density and location of fixed objects relative to the traveled way and steepness of roadside slopes) that cannot be fully explained in the analyses.

In summary, results of the analysis of edgeline rumble strips versus nonedgeline rumble strips compared with no SRS indicated that

- On rural freeways, edgeline rumble strips were more effective in reducing SVROR FI crashes than nonedgeline rumble strips and
- On rural two-lane roads, there was no difference in the safety effect of SRS placed close to the edgeline versus rumble strips placed farther from the edgeline.

## SAFETY EFFECTIVENESS OF CENTERLINE RUMBLE STRIPS

The objectives of the safety evaluation of CLRS were to

- Quantify the safety effectiveness of milled CLRS on specific roadway types including multilane undivided highways and two-lane roads in both rural and urban areas and
- Quantify the safety effectiveness of milled CLRS along varying roadway geometry (i.e., tangent versus horizontal curve).

The safety evaluation investigated the change in crash frequency for TOT crashes, FI crashes, total head-on and sideswipe opposite-direction (TOT target) crashes, and fatal and injury head-on and sideswipe opposite-direction (FI target) crashes.

### Safety Effectiveness of CLRS on Different Roadway Types

Data for this evaluation were collected in MN, PA, and Washington (WA). As a result of insufficient data for a number of roadway types, it was decided to focus the safety evaluation of CLRS on the following roadway types:

- Urban two-lane roads and
- Rural two-lane roads.

Furthermore, for urban two-lane roads, a decision was made to analyze data from PA only and not include data from WA because the number of mile years of data from PA far exceeded that for WA.

In addition to the analyses for rural two-lane roads based on data collected from this study, the raw data and calculations from the study done by Persaud et al. for the Insurance Institute of Highway Safety (IIHS) were available to the research team (*9*). Those data consisted of 98 rural two-lane sites in six states with a total of 211 mi (340 km) of CLRS. The data from the current study and the IIHS study were combined and analyzed together to estimate the safety effectiveness of CLRS on two-lane roads.

Table 4 presents the number of before–after sites included in the analyses, the mile years of data before and after installation of the rumble strips, and the results from the EB analyses for urban and rural two-lane roads. Because of space considerations Table 4 does not present information on the SPFs developed for use in the EB calculations. The results for urban two-lane roads are the first published estimates of the safety effectiveness of CLRS on this roadway type.

The following summarizes the most reliable and comprehensive estimates of the safety effectiveness of CLRS on urban two-lane roads (based on results from this research):

- 40% reduction in TOT target crashes (SE = 17) and
- 64% reduction in FI target crashes (SE = 27).

The following summarizes the most reliable and comprehensive estimates of the safety effectiveness of CLRS on rural two-lane roads [combined results from this research and Persaud et al. (*9*)]:

- 9% reduction in TOT crashes (SE = 2),
- 12% reduction in FI crashes (SE = 3),
- 30% reduction in TOT target crashes (SE = 5), and
- 44% reduction in FI target crashes (SE = 6) (results from this research).

### Safety Effectiveness of CLRS Along Varying Roadway Geometry

This safety evaluation investigated the potential difference in safety effectiveness between CLRS installed along tangents and those installed along horizontal curves. The analysis was based on data for rural two-lane roads in MN, PA, and WA.

Table 5 presents the number of before–after sites included in the analyses, the mile years of data before and after installation of the rumble strips, and the results from the EB analyses separately for tangent and horizontal curve sites, for three crash types of interest. The analyses of TOT and FI crashes do not reveal any insights on this issue; however, the safety benefits of CLRS on horizontal curves and tangents, based on TOT target crashes, are comparable with estimated 47% and 49% reductions in TOT target crashes, respectively. Results for TOT target crashes for WA indicate a 67% reduction in TOT target crashes, but on the basis of the combined results, the analysis indicated that the safety effectiveness of CLRS is, for all practical purposes, the same for curved and tangent alignments.

## VEHICLE DYNAMICS NOISE STUDY

To determine optimum design dimensions for shoulder and centerline rumble strip patterns for a range of operating conditions, a field study was conducted to collect noise data and develop statistical models to predict noise responses in the compartment of a passenger car while it is traversing rumble strips. The statistical models can be used to design milled rumble strip patterns that produce desirable alerting properties. The field study involved driving a midsized passenger car (i.e., Chevrolet Impala) over various rumble strip patterns at different speeds and departure angles. To collect the noise data, a portable data acquisition system was built that used a laptop computer, a global positioning system, a sound level meter (SLM), and a USB analog-to-digital converter module. The SLM was mounted in the vehicle at approximately the same height and fore-aft location as a driver's right ear. The test vehicle was driven at speeds ranging from approximately 40 to 65 mph (65 to 105 km/h). The roadway departure angles collected during testing ranged from 1° to nearly 10°. Data were collected in six states [Arizona (AZ), Colorado (CO), Kentucky (KY), MN, PA, and Utah (UT)], for a total of 990 sound level measurements.

Ordinary least squares (OLS) linear regression was used to develop statistical models to predict sound level differences, defined as the difference between the maximum and ambient sound levels generated during each test. The OLS estimator assumes the following:

- Explanatory nonstochastic variables,
- No omitted or irrelevant variables included in the model specification,
- Disturbance having a mean value of zero and being normally distributed,
- Homoscedastic disturbances,
- No autocorrelation between disturbances,
- No perfect multicollinearity,
- Correctly specified model, and
- Zero covariance between the disturbance and explanatory variables.

Violating the assumptions of the OLS estimator can result in biased, inconsistent, or inefficient parameter estimates. As such, several diagnostic measures were applied to test the OLS assumptions. The Anderson–Darling test was used to test the normality assumption of the disturbances. This test compares the cumulative distribution of the residuals to those of a theoretically normal distribution. The null hypothesis is that the residuals follow a normal distribution. A Breusch–Pagan/Cook–Weisburg test was used to assess the residuals for heteroscedasticity. The null hypothesis

**TABLE 4 Safety Effectiveness of Centerline Rumble Strips on Roadway Types Using the EB Method**

| | | | | | Crash Type | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | TOT | | | FI | | | TOT Target | | | FI Target | | |
| Roadway Type | State | Number of Sites | Mile Years Before | Mile Years After | Change in Crash Frequency from Before to After Rumble Strip Installation (%) Estimate[a] | SE[b] | Test Statistic[c] | Change in Crash Frequency from Before to After Rumble Strip Installation (%) Estimate[a] | SE[b] | Test Statistic[c] | Change in Crash Frequency from Before to After Rumble Strip Installation (%) Estimate[a] | SE[b] | Test Statistic[c] | Change in Crash Frequency from Before to After Rumble Strip Installation (%) Estimate[a] | SE[b] | Test Statistic[c] |
| Urban two-lane roads | Pennsylvania | 74 | 139 | 91 | 1.5 | 8.0 | 0.19 | –9.3 | 9.5 | 0.98 | –40.2 | 17.0 | 2.36 | –63.7 | 26.9 | 2.36 |
| Rural two-lane roads | Combined | 962 | 2,197 | 1,380 | –4.1 | 2.6 | 1.58 | –9.4 | 3.5 | 2.57 | –37.0 | 5.3 | 6.98 | –44.5 | 6.4 | 6.98 |
| | Minnesota | 301 | 1,033 | 604 | –11.1 | 5.8 | 1.91 | –21.8 | 6.6 | 3.30 | –48.9 | 7.3 | 6.69 | –44.7 | 6.7 | 6.69 |
| | Pennsylvania | 526 | 893 | 722 | –1.6 | 3.3 | 0.48 | –6.2 | 4.2 | 1.48 | –25.8 | 17.9 | 1.44 | –44.4 | 30.8 | 1.44 |
| | Washington | 135 | 191 | 54 | 2.3 | 8.1 | 0.28 | 4.1 | 14.6 | 0.28 | –35.4 | 29.2 | 1.21 | –35.4 | 29.2 | 1.21 |
| | IIHS | 98 | 1,122 | 572 | –14.1 | 3.0 | 4.7 | –15.5 | 4.5 | 3.4 | –21.4 | 7.8 | 2.7 | | | |
| | Combined with IIHS | 1,060 | 3,239 | 1,952 | –8.7 | 2.0 | 4.3 | –11.7 | 2.8 | 4.2 | –30.2 | 4.5 | 6.7 | | | |

[a]A negative percent change indicates a decrease in crash frequency, while a positive percent change indicates an increase in crash frequency.
[b]SE = standard error of estimate.
[c]Test statistic = abs(estimate/SE); not significant at 90% CL if <1.7; significant at 90% CL if ≥1.7; significant at 95% CL if ≥2. A row is shaded in gray whenever a percentage change from the analysis shows a statistically significant rumble strip effect at the 95% or 90% confidence level.

TABLE 5 Safety Effectiveness of CLRS Along Varying Geometry on Rural Two-Way Roads Using the EB Method

| | | | | Crash Type | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | TOT | | | FI | | | TOT Target | | |
| Curve Type | State | Number of Sites | Total Length (mi) | Change in Crash Frequency from Before to After Rumble Strip Installation (%) Estimate[a] | SE[b] | Test Statistic[c] | Change in Crash Frequency from Before to After Rumble Strip Installation (%) Estimate[a] | SE[b] | Test Statistic[c] | Change in Crash Frequency from Before to After Rumble Strip Installation (%) Estimate[a] | SE[b] | Test Statistic[c] |
| Horizontal curves | Combined | 331 | 68.15 | 3.5 | 6.5 | 0.54 | –6.4 | 8.1 | 0.79 | –47.1 | 9.9 | 4.76 |
| | Minnesota | 135 | 28.41 | –17.1 | 9.6 | 1.78 | –36.7 | 11.6 | 3.16 | –52.1 | 13.6 | 3.83 |
| | Pennsylvania | 144 | 29.32 | 16.0 | 9.2 | 1.74 | 9.8 | 11.4 | 0.86 | –46.9 | 13.9 | 3.37 |
| | Washington | 62 | 10.42 | 2.7 | 16.0 | 0.17 | –20.7 | 12.9 | 1.60 | 45.5 | 102.9 | 0.44 |
| Tangent | Combined | 312 | 215.24 | –8.0 | 4.3 | 1.86 | –14.9 | 5.9 | 2.53 | –49.3 | 6.9 | 7.14 |
| | Minnesota | 166 | 153.44 | –9.7 | 5.5 | 1.76 | –17.8 | 7.8 | 2.28 | –48.8 | 7.9 | 6.18 |
| | Pennsylvania | 73 | 23.25 | –9.9 | 8.4 | 1.18 | –21.8 | 10.0 | 2.18 | –43.3 | 18.4 | 2.35 |
| | Washington | 73 | 38.55 | 2.0 | 9.3 | 0.22 | 10.2 | 17.3 | 0.59 | –67.3 | 19.0 | 3.54 |

[a]A negative percent change indicates a decrease in crash frequency, while a positive percent change indicates an increase in crash frequency.
[b]SE = standard error of estimate.
[c]Test statistic = abs(estimate/SE); not significant at 90% CL if <1.7; significant at 90% CL if ≥1.7; significant at 95% CL if ≥2. A row is shaded in gray whenever a percentage change from the analysis shows a statistically significant rumble strip effect at the 95% or 90% confidence level.

($\chi^2$ test) is that the residuals have a constant variance. The autocorrelation assumption was tested using the Durbin–Watson statistic. The null hypothesis is that the residuals are not autocorrelated. Variance inflation factors (VIFs) were used to determine the presence of multicollinearity. VIFs are a measure of multicollinearity among the explanatory variables in a model; values exceeding 10 indicate that multicollinearity is present. The Ramsey RESET test was used to assess the model for omitted variable bias. The null hypothesis ($F$-test) is that the model has no omitted variables. When violations of the model assumptions were identified, various treatments were applied.

The dependent variable used in the model specification is the sound level difference. The sound level difference was computed as the difference between the maximum sound level generated as the test vehicle traversed the rumble strip pattern minus the ambient sound generated in the passenger compartment of the test vehicle before encroaching the rumble strips. Separate models for the maximum and ambient sound levels were not specified because the sound level distributions obtained on the travel lane (ambient sound) and as the vehicle traversed the rumble strip pattern (maximum sound) were different. Specifically, the variability of these distributions differs; therefore, using predictions from separate ambient and maximum sound level models will either over- or underestimate the sound level difference experienced by drivers that leave the roadway and pass over a rumble strip pattern. A model of the sound level difference is based only on a single distribution.

The general form of the model was as follows:

$$SLdiff_i = \beta X_i + \epsilon_i$$

where

$SLdiff_i$ = sound level difference (dBA) for rumble strip $i$,
$\beta$ = regression parameter estimates,
$X_i$ = vector of explanatory variables for rumble strip $i$, and
$\epsilon_i$ = disturbance term for rumble strip $i$.

Table 6 presents two models that predict the sound level difference generated by rumble strips in the compartment of a passenger car. One model includes state indicator variables; the other does not. These are the first predictive models to date that include all four primary rumble strip dimensions (i.e., length, width, depth, and spacing). A model from previous research by Khan and Bacchus included only rumble strip width and depth as significant predictors of in-vehicle sound, in addition to vehicle speed (*10*).

The adjusted $R^2$ values of the models are relatively low. Several possible explanations for these low values are as follows:

- Vehicle tires used on the test vehicles may have had different inflation pressures or tread wear.
- Pavement surface temperatures were different among and within test locations, which influenced the dynamic interaction between the tire and pavement surface.
- Rumble strip pattern wear differs in test locations, particularly on asphalt pavement surfaces.
- Pavement surface texture can vary considerably between test locations.

Agencies may use either of the two noise prediction models presented in Table 6 to establish rumble strip dimensions for a range of rumble strip applications. An example is provided below to demonstrate their use. When applying the model with the state indicator variables, unless an agency is from one of the four states represented in the model, it is recommended that the agency assume the base conditions.

On the basis of a review of the literature and current practice, to determine the minimum level of stimuli generated by a rumble strip to be able to alert an inattentive, distracted, drowsy, or fatigued driver, it is recommended that on roadways on which bicyclists are not expected, rumble strip patterns should be designed to generate approximately 10 to 15 dBA above the ambient in-vehicle sound level; whereas for roadways on which bicyclists may be expected, it is recommended that rumble strip patterns be designed to generate

**TABLE 6 Linear Regression Models to Predict Sound Level Difference in Compartment of Passenger Car While Traversing Rumble Strips**

| Variable | Parameter Estimate | Standard Error | *t*-Statistic | $P>\|t\|$ | 95% Confidence Limits | |
|---|---|---|---|---|---|---|
| | | | | | Lower | Upper |
| Model Without State Indicator Variables[a] | | | | | | |
| Constant | 8.650 | 1.619 | 5.34 | <.001 | 5.474 | 11.827 |
| Speed (mph) | 0.027 | 0.018 | 1.48 | .140 | –0.009 | 0.063 |
| Location indicator (1: shoulder; 0: centerline) | –1.689 | 0.398 | –4.25 | <.001 | –2.470 | –0.909 |
| Angle of departure (degrees) | –0.271 | 0.076 | –3.55 | <.001 | –0.421 | –0.121 |
| Length (in.) | 0.267 | 0.029 | 9.35 | <.001 | 0.211 | 0.323 |
| Width (in.) | 0.771 | 0.126 | 6.13 | <.001 | 0.525 | 1.018 |
| Depth (in.) | 4.494 | 1.328 | 3.38 | .001 | 1.887 | 7.100 |
| Spacing (in.) | –0.394 | 0.041 | –9.70 | <.001 | –0.474 | –0.314 |
| Rumble strip type indicator (1: milled; 0: rolled) | 2.652 | 0.811 | 3.27 | .001 | 1.060 | 4.244 |
| Pavement surface type indicator (1: concrete; 0: asphalt) | –1.391 | 0.685 | –2.03 | .043 | –2.736 | –0.046 |
| Pavement surface condition indicator (1: wet; 0: dry) | –2.596 | 0.465 | –5.58 | <.001 | –3.509 | –1.683 |
| Model with State Indicator Variables[b] | | | | | | |
| Constant | 7.712 | 1.291 | 5.97 | <.001 | 5.178 | 10.246 |
| Speed (mph) | 0.057 | 0.016 | 3.58 | <.001 | 0.026 | 0.089 |
| Location indicator (1: shoulder; 0: centerline) | –1.116 | 0.335 | –3.34 | <.001 | –1.773 | –0.459 |
| Angle of departure (degrees) | –0.275 | 0.079 | –3.50 | <.001 | –0.429 | –0.121 |
| Length (in.) | 0.352 | 0.037 | 9.45 | <.001 | 0.279 | 0.425 |
| Width (in.) | 0.498 | 0.191 | 2.61 | .009 | 0.123 | 0.873 |
| Depth (in.) | 3.106 | 1.495 | 2.08 | .038 | 0.172 | 6.041 |
| Spacing (in.) | –0.300 | 0.050 | –5.97 | <.001 | –0.398 | –0.201 |
| Pennsylvania indicator[c] (1: Pennsylvania; 0: otherwise) | 2.197 | 0.513 | 4.28 | <.001 | 1.189 | 3.205 |
| Minnesota indicator[c] (1: Minnesota; 0: otherwise) | 1.165 | 0.752 | 1.55 | .122 | –0.310 | 2.641 |
| Arizona indicator[c] (1: Arizona; 0: otherwise) | 4.039 | 0.713 | 5.66 | <.001 | 2.639 | 5.439 |
| Utah indicator[c] (1: Utah; 0: otherwise) | –3.219 | 0.937 | –3.43 | .001 | –5.058 | –1.379 |
| Pavement type indicator (1: concrete; 0: asphalt) | –3.065 | 0.568 | –5.40 | <.001 | –4.179 | –1.950 |

[a]Number of observations: 990; $R^2$: 0.179; $R^2_{adj}$: 0.171; and root-mean-square error: 3.936.
[b]Number of observations: 850; $R^2$: 0.332; $R^2_{adj}$: 0.323; and root-mean-square error: 3.452.
[c]Kentucky and Colorado are the baseline, and set to zero. State effects in the table should be interpreted using Kentucky and Colorado as a baseline. State effects with a negative sign are expected to have lower sound level differences than the baseline, while state effects with a positive sign are expected to have higher sound level differences than the baseline.

between 6 and 12 dBA above the ambient in-vehicle sound level. Another way to consider this is that for roadways on which bicyclists are not expected, more aggressive rumble strip patterns may be used; whereas for roadways on which bicyclists are expected, more bicycle-tolerable rumble strip patterns should be used. Again, these design guidelines are based on a review of stimuli levels recommended to alert drivers, covering a range of disciplines.

## Sample Application of Sound Level Difference Models

Suppose a transportation agency wants to establish a policy for the design of milled SRS on rural and urban freeways with posted speed limits between 55 and 65 mph (88 and 105 km/h). On freeways, shoulders may consist of either concrete or asphalt pavement. Horizontal curves on freeways are relatively flat because of the higher design speeds, so the angle of roadway departures can be assumed to be relatively low. Because wet pavement surfaces produce lower sound level differentials than dry pavement surfaces, a wet pavement surface will be assumed. Freeways typically have wide shoulders, and in most cases bicycles will not be permitted; thus the length dimension of the rumble strip can be determined independently of shoulder width and bicycle considerations and more aggressive rumble strips can be designed. Also, because bicycles are not permitted on freeways in most states, the rumble strips can be designed for the higher ranges of desirable sound level difference. Since asphalt surfaces generate higher sound level differences than concrete surfaces, asphalt surfaces will be considered in the analysis.

This example uses the noise prediction model without the state indicator variables to establish potential rumble strip dimensions. Rumble strip dimensions for the right (outside) shoulder of the freeway will be considered here. The process could be repeated for establishing desirable dimensions for the left (median) shoulder of the freeway. In this scenario, the following information is known:

- Location: right (outside) shoulder rumble strips (indicator = 1),
- Rumble strip type: milled (indicator = 1),

- Pavement type: asphalt (indicator = 0), and
- Pavement condition: wet (indicator = 1).

Inputting these values into the sound level difference model yields the following:

$$SLdiff = 7.017 + 0.027\ speed - 0.271\ angle + 0.267\ length + 0.771\ width + 4.494\ depth - 0.394\ spacing$$

The dimensions for three potential rumble strip patterns, two vehicle speed levels [assuming 55 and 65 mph (88 and 105 km/h) posted speeds], and three angles of departure are shown in Table 7. Pattern 1 can be assumed to be an edgeline rumble strip; Patterns 2 and 3 are common shoulder rumble strip patterns used in Canada and the United States, respectively. Assume for this example that a transportation agency would like to develop rumble strip patterns that generate sound level differences in the range of 10 to 15 dBA (i.e., 10 dBA ≤ SLDiff ≤ 15 dBA). The last column in Table 7 shows the estimated difference in sound level generated in the compartment of a passenger car by the three rumble strip patterns based on the given design parameters. The shaded rows represent designs in which the rumble strip pattern, speed, and angle of departure fall outside the desired sound level difference range. For a departure angle of 1°, Patterns 1 and 2 are expected to generate the desirable sound level difference for both speed ranges. When the departure angle is increased to 3°, Patterns 1 and 2 are again expected to produce sound level differences in the desired range; Pattern 3 would be expected to generate sound levels slightly below the maximum criterion for speeds of 55 mph (88 km/h) and slightly above the maximum criterion for speeds of 65 mph (105 km/h). For a departure angle of 5°, Rumble Strip Patterns 2 and 3 are expected to produce sound levels in the desirable range at both speeds [55 and 65 mph (88 and 105 km/h)], whereas Pattern 1 would be expected to generate slightly less than the desired sound level. From this example, Pattern 2 appears to be the most desirable pattern on the basis of the agency's policy decision, but Patterns 1 and 3 are close to meeting the desired design guidelines so it is conceivable that Patterns 1 and 3 would also be acceptable for use by the transportation agency along urban and rural freeways.

## Bicycle Considerations

Several transportation agencies would prefer to install SRS with groove lengths less than the typical 12 to 16 in. (305 to 406 mm) to keep this dimension as narrow as possible to provide additional lateral clearance on roadway shoulders for bicyclists. Designing an SRS with a narrow groove length also allows the installation of rumble strips on roadways with narrow or nonexistent shoulders on which rumble strips with typical groove lengths could not otherwise be installed. With all other dimensions equal, there is approximately a 2.5- to 3.6-dBA difference in sound level above the ambient when comparing a rumble strip with a 6-in. (152-mm) length with a rumble strip with a length of 16 in. (406 mm). Considering that the range of recommended design thresholds for roadways on which bicyclists may be expected is 6 dBA (i.e., an increase between 6 to 12 dBA above the ambient sound level is desirable), a rumble strip with a 16-in. (406-mm) length that is expected to generate an increase above ambient sound between 9 and 12 dBA can be reduced in length to 6 in. (152 mm) and still be expected to generate a sound level difference above the minimum desirable level of 6 dBA. Thus, it can be concluded that rumble strips designed with narrower lengths [e.g., 6 in. (152 mm)] can generate the desired sound level differences to alert inattentive, distracted, drowsy, or fatigued drivers. It is not necessarily recommended that all rumble strip patterns have a narrow length of 6 in. (152 mm), but in those situations in which it is desirable to design a rumble strip with a narrower length for a particular reason (e.g., to provide more lateral clearance for bicyclists or for very narrow shoulders), rumble strip patterns can be designed as such and still be effective.

TABLE 7 Rumble Strip Dimensions and Parameters Considered in Sample Application of Noise Prediction Model

| Rumble Strip Pattern | Rumble Strip Dimensions | | | | Speed (mph) | Departure Angle (degrees) | Sound Level Difference (dBA) |
|---|---|---|---|---|---|---|---|
| | Length (in.) | Width (in.) | Depth (in.) | Spacing (in.) | | | |
| 1 | 6 | 5 | 0.375 | 12 | 55 | 1 | 10.70 |
| 2 | 12 | 6 | 0.375 | 12 | 55 | 1 | 13.09 |
| 3 | 16 | 7 | 0.5 | 12 | 55 | 1 | 15.51 |
| 1 | 6 | 5 | 0.375 | 12 | 55 | 3 | 10.16 |
| 2 | 12 | 6 | 0.375 | 12 | 55 | 3 | 12.55 |
| 3 | 16 | 7 | 0.5 | 12 | 55 | 3 | 14.97 |
| 1 | 6 | 5 | 0.375 | 12 | 55 | 5 | 9.62 |
| 2 | 12 | 6 | 0.375 | 12 | 55 | 5 | 12.58 |
| 3 | 16 | 7 | 0.5 | 12 | 55 | 5 | 14.43 |
| 1 | 6 | 5 | 0.375 | 12 | 65 | 1 | 10.97 |
| 2 | 12 | 6 | 0.375 | 12 | 65 | 1 | 13.93 |
| 3 | 16 | 7 | 0.5 | 12 | 65 | 1 | 15.78 |
| 1 | 6 | 5 | 0.375 | 12 | 65 | 3 | 10.43 |
| 2 | 12 | 6 | 0.375 | 12 | 65 | 3 | 13.39 |
| 3 | 16 | 7 | 0.5 | 12 | 65 | 3 | 15.24 |
| 1 | 6 | 5 | 0.375 | 12 | 65 | 5 | 9.89 |
| 2 | 12 | 6 | 0.375 | 12 | 65 | 5 | 12.85 |
| 3 | 16 | 7 | 0.5 | 12 | 65 | 5 | 14.70 |

## CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE RESEARCH

The conclusions from this research were as follows:

- Shoulder and centerline rumble strips are effective low-cost crash mitigation measures. From the results of this research, the best available estimates of their safety effectiveness were as follows:
  - SRS on rural freeways are expected to reduce SVROR crashes by 11% and SVROR FI crashes by 16%.
  - SRS on rural two-lane roads are expected to reduce SVROR crashes by 15% and SVROR FI crashes by 29%.
  - CLRS on urban two-lane roads are expected to reduce TOT target crashes by 40% and FI target crashes by 64%.
  - CLRS on rural two-lane roads are expected to reduce TOT crashes by 9%, FI crashes by 12%, TOT target crashes by 30%, and FI target crashes by 44%.
- The lack of reliable estimates on the safety effectiveness of shoulder and centerline rumble strips for other roadway types does not necessarily mean they are ineffective on these roadway types; rather, the safety effects of shoulder and centerline rumble strips on these other facility types could not be evaluated satisfactorily at this time.
- On rural freeways, SRS should be placed as close to the edgeline as possible to maximize their safety benefits while also taking into consideration other factors (e.g., pavement joints). For other roadway types, such as rural two-lane roads, there is no conclusive evidence to indicate that offset distance affects the safety effectiveness of SRS. Therefore, strictly on the basis of safety benefits, there is no current basis for recommending that transportation agencies change their current policies concerning the placement of SRS relative to the edgeline on these other roadway types.
- The safety benefits of CLRS on horizontal curves and on tangent sections are identical, for all practical purposes.
- Statistical models developed in this research to predict the sound level difference in the passenger compartment of a midsize car when traversing rumble strips can be used to design rumble strip patterns that produce the desired alerting properties. Predictive models are available that include as independent variables the four primary rumble strip dimensions (i.e., length, width, depth, and spacing), vehicle speed, angle of departure, pavement type (asphalt or concrete), pavement condition (wet or dry), rumble strip type (milled or rolled), and location (shoulder or centerline).
- In situations in which it is desirable to provide more lateral clearance for bicyclists or for installing SRS on roads with very narrow shoulders, SRS can be designed with relatively narrow lengths [e.g., 6 in. (152 mm)] and still generate the desired sound level differences in the passenger compartment.

Several key unresolved issues associated with shoulder and centerline rumble strips that should be addressed in the near future include

- Determining the optimal placement of SRS on rural two-lane roads,
- Better quantifying the safety effectiveness of SRS in varying conditions (e.g., along varying roadway geometry),
- Better quantifying the safety effectiveness of CLRS in varying conditions (i.e., along passing zones and no-passing zones),
- Determining the minimum level of stimuli (i.e., sound or vibration) necessary to alert a drowsy or inattentive driver,
- Determining the effect of rumble strips on pavement performance, and
- Assessing the effect of noise produced by rumble strips on adjacent residents.

Several key unresolved issues that should be addressed in the longer term (i.e., allow another 3 to 5 years for more rumble strip installations) include

- Better quantifying the safety effectiveness of SRS on different roadway types,
- Better quantifying the safety effectiveness of CLRS on different roadway types, and
- Quantifying the safety effectiveness of dual application treatments (i.e., shoulder and centerline rumble strips installed along the same roadway).

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# Do Pedestrian Countdown Signals Influence Vehicle Speeds?

Shashi S. Nambisan and Ganesh J. Karkee

**The effects of pedestrian countdown signals on speeds of approaching vehicles as a function of the distance of the vehicle upstream of the intersection and the signal display are reported here. The analyses used mean speeds of vehicles on two segments immediately upstream of the stop bar and also during different indications of the pedestrian signal head (pedestrian "Walk" and pedestrian flashing "Don't Walk" signal with the time remaining in seconds to cross the street for the following intervals: greater than 15 s, 15 s to 10 s, 10 s to 5 s, and less than 5 s). Appropriate tests were performed to determine the statistical significance of the differences in observed mean speeds for each of these scenarios. Vehicle speeds were greater on the segment closer to the intersection than on the segment farther away. The speeds were also greater during the pedestrian countdown timer and flashing "Don't Walk" displays on the pedestrian signal head than when the pedestrian "Walk" signal and the countdown timer were displayed. All results were statistically significant at a 95% level of confidence except one, which had a 93% level of confidence. Results do not indicate that speeds are affected by the actual numeric displays on the countdown timer.**

During the past decade or so, several agencies have opted to replace the conventional pedestrian signal heads with pedestrian countdown signals. A pedestrian interval countdown display is added to a pedestrian signal head to inform pedestrians of the number of seconds remaining in the pedestrian change interval (*1*). The pedestrian countdown signals aim to help pedestrians to better judge the remaining time to cross the roadway. On these signal heads, when the allocated time for the "Walk" signal is completed, time remaining for the pedestrian to complete the crossing maneuver is displayed (in seconds in Arabic numerals) along with the flashing "Don't Walk" signal. The time remaining is displayed in a descending order, that is, it is a countdown. In addition to being visible to pedestrians in the proximity of the crosswalk, these countdown timers are also visible to motorists approaching the intersection. Motorists on approaches parallel to the crosswalk with operational countdown timers also have a green signal for their movement through the intersection. The time displayed on the countdown signal head is also visible to these motorists. A view of the countdown clock indication on the pedestrian signal head could affect motorists' speeds when compared with those at signal heads with the conventional pedestrian signals with only the flashing "Don't Walk" indications. There is concern that motorists may look to these displays to increase their speeds to try to get through the intersection, rather than having to wait through another signal cycle to complete their movement through the intersection. Although many studies have addressed that question, it warrants further attention because the studies tend to be based solely on speeds.

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## RELEVANT LITERATURE

A literature review finds several efforts to evaluate the pedestrian countdown signal. Several studies on the effectiveness of pedestrian countdown signals in the United States have been documented in the literature. Many of these studies are focused on evaluation of the pedestrian countdown timers on the basis of pedestrian behavior (*2–6*). Other studies are related to devices in enhancing pedestrian safety (*7–14*). These studies evaluated pedestrians' crossing behavior on the basis of different signs or with different messages displayed on the countdown signals. Following is a summary of the literature review from studies related to pedestrian and driver behavior toward pedestrian countdown signals.

Eccles et al. evaluated the pedestrian countdown signals in Montgomery County, Maryland. A before and after study technique was used to evaluate motorists' and pedestrians' behavior and vehicle speed. No effect on vehicle approach speed was observed resulting from the presence of countdown signals while vehicles entered intersections during clearance intervals (*15*). In a study on driver behavior during flashing green before amber, Koll et al. found that the flashing green increases the number of early stops (*16*). Drivers tend to underestimate the flashing green duration. These early stops are not compliant with the rules of the road, which produces a large option zone. Drivers can both safely stop and cross at an option zone. The large option zone generates a period of uncertainty, in which a trailing driver cannot easily predict whether the leading driver will stop or cross.

Huey and Ragland conducted a study on changes in driver behavior due to pedestrian countdown signals (*17*). Observations were made at two intersections, one with pedestrian countdown signals and another without. The main focus of the study was driver behavior during the red and amber phase. The authors found that at the end of the amber phase drivers were less likely to enter the intersection with a pedestrian countdown signal compared with an intersection with a traditional pedestrian signal. A significantly higher number of vehicles are present as the amber phase expires at intersections with traditional pedestrian signals, which would have potentially adverse effects on safety. Schattler et al. studied the effects of countdown signals on pedestrian and motorist behavior at 13 intersections in Peoria,

Illinois (*18*). Measures of effectiveness used in this study were the proportion of pedestrians who started crossing the street during the "Walk," flashing "Don't Walk," and steady "Don't Walk" intervals and the occurrence of vehicles entering the intersection late in the yellow interval or after the onset of the red signal indication. The authors report that pedestrian movement data indicated that the use of countdown displays increases compliance with pedestrian signals. The red and yellow light running characteristics indicated no difference between before and after periods or between test and control sites. The installation of pedestrian countdown signals cannot be attributed to increased risk-taking behavior on the part of motorists.

Bundy and Schrock conducted a comparative study at four sites, two with countdown timers and two without (*19*). Speed data were collected for approaching traffic between 400 ft upstream from the intersection and the intersection stop bar. They found that drivers were significantly less likely to increase their speed to reach the intersection before the beginning of the red phase when countdown timers were present, and some drivers began to slow to stop before the beginning of the amber phase. Markowitz et al. evaluated 14 intersections with pedestrian countdown timers in San Francisco (*20*). Countdown signals appeared to reduce pedestrian injuries. The countdown timers reduced the proportion of pedestrians finishing crossing on the red. The countdown did not result in an increase in drivers running red lights. Lum and Halim report that a green signal countdown was effective in encouraging red light stopping actions by motorists, but not curbing red light running violations (*21*).

## APPROACH

Studies cited in the literature typically used before and after studies to evaluate the effect of countdown timers on vehicle speeds. However, this paper addresses the speeds of approaching vehicles at different distances upstream of the signal in comparison with the posted speed limit for the approach roadway segment, and as a function of the display on the countdown signal head. The conjecture here is that drivers wishing to get through an intersection without stopping for a red phase may increase their vehicle speed (if it is not hindered by other vehicles) during the approach to the intersection so as to legally go through the intersection. To examine this conjecture the following pilot study was used.

For the pilot investigation, a segment 200 ft (61 m) from the stop bar on the upstream of the intersection was measured to obtain space mean speed. To help visual observations of location and time measurements, an identifying mark was established (on the sidewalk) at the midpoint of the segment that resulted in the creation of two segments on either side, each 100 ft (30.5 m) in length. The segment farther upstream from the intersection is called the first segment, and the other is called the second segment. The time taken by a vehicle to travel each of these segments was recorded. Three observers were located on the sidewalk at separate locations. The first observer, stationed 200 ft (61 m) upstream of the stop bar, signaled when a vehicle to be observed crossed the landmark. The second observer stationed in the middle of the 200-ft (61-m) section waited for the signal from the first observer to record the travel time using two stopwatches (accurate to one-tenth of a second). When the subject crossed the landmark two stopwatches were turned on simultaneously. Once the subject vehicle entered the second segment one stopwatch was stopped. When the subject vehicle crossed the stop bar, the second stopwatch was also stopped and the corresponding times were recorded. The third observer, who was stationed at the stop bar mark on the sidewalk, recorded the displayed information on the countdown timer when the subject vehicle crossed the landmark 200 ft (61 m) upstream of the stop bar.

Motorists will have sufficient time to react to a change in the signal indication (stimulus) while traversing the first segment. Therefore, in response to the stimulus, they might speed up or slow down depending on events at the intersection. The time taken to traverse the first segment is denoted as $t_1$ s and the corresponding speed is $V_1$ km/h. Similarly, $t_2$ and $V_2$ are the time (in s) and speed (in km/h) for the second segment. The corresponding indications on the pedestrian signal head were also noted as one of the following intervals: either pedestrian "Walk" or countdown timings (>15 s, 15–10 s, 10–5 s, <5 s), and the speed of the vehicle was computed (*22*).

The two speeds, $V_1$ and $V_2$, were compared for various time intervals as denoted by the countdown signal in the pedestrian signal head, including the pedestrian "Walk" signal. The vehicle speeds during the various time intervals (based on the pedestrian signal head indication) were compared to see whether there were any differences in speed that could be attributed to the pedestrian countdown signal. A paired *t*-test was used at the 95% confidence level.

### Two-Sample *t*-Test

The speeds, $V_1$ and $V_2$, were compared when pedestrian "Walk" signals and countdown numerals were displayed on the pedestrian signal head. A two-sample *t*-test, the Welch–Satterthwaite *t*-test, was used to compare the speeds and to determine whether they were statistically different at the 95% confidence level.

The Welch–Satterthwaite *t*-test is used when the assumption that the two populations have equal variances seems unreasonable. It provides a *t*-statistic that asymptotically approaches a *t*-distribution as the sample sizes become large, allowing for an approximate *t*-test to be calculated when the population variances are not equal. This test is different from the ordinary Student's *t*-distribution. The variances of the two groups are assumed equal for the Student's *t*-distribution (*23*). The Welch's *t*-test is used to identify the difference between means of independent samples.

Let

$\mu_W$ = population mean speed of vehicles during the pedestrian "Walk" signal,
$n_W$ = number of observations during pedestrian "Walk" signal,
$\bar{x}_W$ = sample mean of $n_W$ observations, and
$s_w^2$ = sample variance of $n_W$ observations.

Similarly, $\mu_C$, $n_C$, $\bar{x}_C$, and $s_C^2$ are the population mean, number of observations, sample mean, and sample variance of speeds, respectively, while the pedestrian countdown timer is displayed on the pedestrian signal head.

The null hypothesis is that means of speeds with "pedestrian 'Walk' signal" and "countdown timer" are equal. The alternate one-sided hypothesis is that the mean of speeds during the countdown timer display is greater than the mean of speeds during the walk phase. These are expressed as follows:

$$H_0: \mu_C - \mu_W = 0$$

$$H_a: \mu_C - \mu_W > 0$$

The test statistic computed from the sample is

$$t_0 = \frac{\bar{x}_C - \bar{x}_W}{\sqrt{\frac{s_C^2}{n_C} + \frac{s_W^2}{n_W}}} \quad (1)$$

The distribution of the test statistic when $H_0$ is true is a $t$-distribution with approximate degrees of freedom (df) given by the following (*24*):

$$\text{df} = \frac{\left(\frac{s_C^2}{n_C} + \frac{s_W^2}{n_W}\right)^2}{\frac{\left(\frac{s_C^2}{n_C}\right)^2}{n_C - 1} + \frac{\left(\frac{s_W^2}{n_W}\right)^2}{n_W - 1}} \quad (2)$$

The significance probability or $P$-value for equality of means versus the one-sided alternative is calculated by

$$P\text{-value} = P(t_{\text{df}} > t_0)$$

If the $P$-value obtained is greater than the critical $\alpha$-value, that is, .05 at the 95% confidence level, then $H_0$ is accepted. Similarly, if the $P$-value is less than the $\alpha$-value, then $H_0$ is rejected at the 95% confidence level.

### Paired *t*-Test

A paired $t$-test was conducted to test the motorists' speeds at different intervals of interest. A set of $n$ paired speed observations was made for the first and second segments, in which the mean and variance of observations on the first segment are represented by $\mu_1$ and $\sigma_1^2$, respectively, and $\mu_2$ and $\sigma_2^2$ are the mean and variance for observations made on the second segment. A one-sample $t$-test on the difference between paired means $\mu_D$ ($\mu_D = \mu_2 - \mu_1$) is used to test the hypotheses about the difference between $\mu_1$ and $\mu_2$ (*25*). The null hypothesis is that there is no difference in speeds at the two locations and the one-sided alternative that the speeds on the segment are greater than the speeds on the first segment is expressed as

$$H_0: \mu_D = 0$$

$$H_a: \mu_D > 0$$

The test statistic computed from the sample is

$$t_0 = \frac{\bar{D}}{S_D\sqrt{n}} \quad (3)$$

where $\bar{D}$ and $S_D$ are the mean and standard deviation of difference of the speed observations. The significance probability or $P$-value is calculated by

$$P\text{-value} = P(t_{\text{df}} > t_0)$$

As stated previously, if the $P$-value calculated is more than the critical $\alpha$-value, that is, .05 at the 95% confidence level, then $H_0$ is accepted. Similarly, if the $P$-value is less than the $\alpha$-value, then $H_0$ is rejected at the 95% confidence level.

These hypotheses were tested with data from field observations conducted in Henderson, Nevada.

### Site Description

Two intersections, Green Valley Parkway and Warm Springs Road and Sunset Road and Marks Street, were used to evaluate the hypotheses presented. These two sites are located in the city of Henderson, which is in the southeastern part of the Las Vegas metropolitan area. The streets at these intersections had posted speed limits of 35 mph (56.4 km/h) on the minor streets, and 40 mph (64.4 km/h) and 45 mph (72.5 km/h), respectively, on the major streets. Data were collected in July and August 2005. The estimated average daily traffic (ADT) and the posted speed limits on each site are shown in Table 1 (*26*).

## RESULTS

A summary of the speeds observed at the study sites is presented in Table 2. At both sites the observed speeds were close to the posted speed limit. At Site 1 (Green Valley Parkway and Warm Springs Road) the mean speeds in the interval when the pedestrian countdown timer was displayed on the pedestrian signal head were 6.1 km/h (3.8 mph) and 4.4 km/h (2.7 mph) greater than those of the mean speeds with the pedestrian "Walk" signals for the eastbound and westbound direction, respectively. Similar differences were noted at Site 2 (Sunset Road and Marks Street) as well: the mean speeds during the pedestrian countdown timer interval were 4.0 km/h (2.5 mph) and 4.6 km/h (2.9 mph) greater than those of the mean speeds with the pedestrian "Walk" signal in the eastbound and westbound direction, respectively. These speed differences are all statistically significant at the 95% confidence level, except the eastbound approach at Sunset Road and Marks Street for which it is significant at a 90% level of confidence ($p = .070$).

Table 3 and Table 4 summarize the results at the two sites of the observed vehicle speeds on the two segments identified immediately upstream of the intersection, along with results of the statistical analyses. The mean speeds of vehicles with pedestrian "Walk" signal displayed on the pedestrian signal head and with the four intervals of the countdown timer displayed on the pedestrian

**TABLE 1 ADT, Speed Limits, and Lane Configuration at Study Sites**

| Location | Average Daily Traffic (ADT) | Posted Speed Limit [mph (km/h)] | Lane Configuration |
|---|---|---|---|
| Green Valley Parkway and Warm Springs Road | 31,000; 29,600 | 35 (56.4); 40 (64.4) | (2T + 1(T + R) + 1L); (2T + 1L + 1R) |
| Sunset Road and Marks Street | 48,000; 8,000 | 45 (72.5); 35 (56.4) | (2T + 1L + 1R); (3T + 2L + 1R) |

NOTE: T = through lane, L = left-turn lane, and R = right-turn lane.

TABLE 2 Mean Speeds: Pedestrian "Walk" Signal Versus Countdown Timer

| Location | Mean Speed (km/h) | | Difference in Mean (km/h) | *P*-Value | Null Hypothesis |
|---|---|---|---|---|---|
| | Pedestrian Walk Signal | Countdown Timer | | | |
| Green Valley Parkway and Warm Springs Road (eastbound) | 58.7 (78) | 64.8 (249) | 6.1 | <.001 | Reject |
| Green Valley Parkway and Warm Springs Road (westbound) | 59.9 (48) | 64.3 (145) | 4.4 | .001 | Reject |
| Sunset Road and Marks Street (eastbound) | 62.4 (31) | 66.4 (140) | 4.0 | .070 | Do not reject |
| Sunset Road and Marks Street (westbound) | 57.9 (45) | 62.5 (93) | 4.6 | .005 | Reject |

NOTE: Values in parentheses are sample sizes.

signal head are also shown in Figure 1 and Figure 2 at Green Valley Parkway and Warm Springs Road and Sunset Road and Marks Street, respectively.

At both study sites, $V_2$, the speed on the second segment [from the stop bar to 100 ft (30.5 m) upstream of the stop bar], was greater than $V_1$, speed on the first segment [between 100 ft (30.5 m) and 200 ft (61 m) upstream of the stop bar]. These differences are highly significant ($p < .001$) for all intervals of interest (intervals are based on the indications on the pedestrian signal head including the pedestrian "Walk" signal).

The result indicates that vehicles' speeds when the "Walk" phase is displayed on the pedestrian signal head are nominally lower than their speeds during the numeric countdown display intervals. This difference is more pronounced on Segment 2, when the vehicles are closer to the intersection. However, the speeds appear to be relatively even during the intervals when the countdown timer is displayed on the pedestrian signal head.

The results may be interpreted as motorists probably observe only the "Walk" signal and the flashing "Don't Walk" countdown numeric display. Once the signal on the pedestrian signal head changes from the "Walk" to the countdown numeric and flashing "Don't Walk," motorists tend to speed up no matter what the number is on the pedestrian signal head. Another reason might be that motorists have to focus on several cues while driving so that these numerals on the pedestrian signal head are not of as much significance to motorists or they may not be clearly discernible from a distance.

Related factors that could affect this observed behavior include the signal cycle lengths, which ranged from 120 to 180 s. If the signal cycle lengths are perceived to be "long" by motorists, they are likely to try to speed up to go through an intersection before the onset of the red phase, rather than endure a "long" wait for the next green phase for their movement. Other factors that could play an important role in a driver's decision to adjust speed are the

TABLE 3 Speeds at Green Valley Parkway and Warm Springs Road

| Sign on Pedestrian Signal Head | Sample Size ($n$) | Mean Speed (km/h) | | Difference in Mean ($V_2 - V_1$) (km/h) | *P*-Value | Null Hypothesis |
|---|---|---|---|---|---|---|
| | | 61 m Upstream of Stop Bar ($V_1$) | 30.5 m Upstream of Stop Bar ($V_2$) | | | |
| Pedestrian Walk signal | 126 | 49.4 | 76.2 | 26.8 | <.001 | Reject |
| Countdown time (>15 s) | 141 | 50.4 | 85.1 | 34.7 | <.001 | Reject |
| Countdown time (15–10 s) | 66 | 55.5 | 84.4 | 28.9 | <.001 | Reject |
| Countdown time (10–5 s) | 61 | 53.4 | 81.9 | 28.5 | <.001 | Reject |
| Countdown time (<5 s) | 75 | 54.8 | 84.2 | 29.4 | <.001 | Reject |

TABLE 4 Speeds at Sunset Road and Marks Street

| Sign on Pedestrian Signal Head | Sample Size ($n$) | Mean Speed (km/h) | | Difference in Mean ($V_2 - V_1$) (km/h) | *P*-Value | Null Hypothesis |
|---|---|---|---|---|---|---|
| | | 61 m Upstream of Stop Bar ($V_1$) | 30.5 m Upstream of Stop Bar ($V_2$) | | | |
| Pedestrian Walk signal | 76 | 53.2 | 71.7 | 18.5 | <.001 | Reject |
| Countdown time (>15 s) | 31 | 52.8 | 83.4 | 30.6 | <.001 | Reject |
| Countdown time (15–10 s) | 61 | 55.4 | 75.9 | 20.5 | <.001 | Reject |
| Countdown time (10–5 s) | 64 | 58.2 | 75.1 | 16.9 | <.001 | Reject |
| Countdown time (<5 s) | 77 | 54.7 | 79.5 | 24.8 | <.001 | Reject |

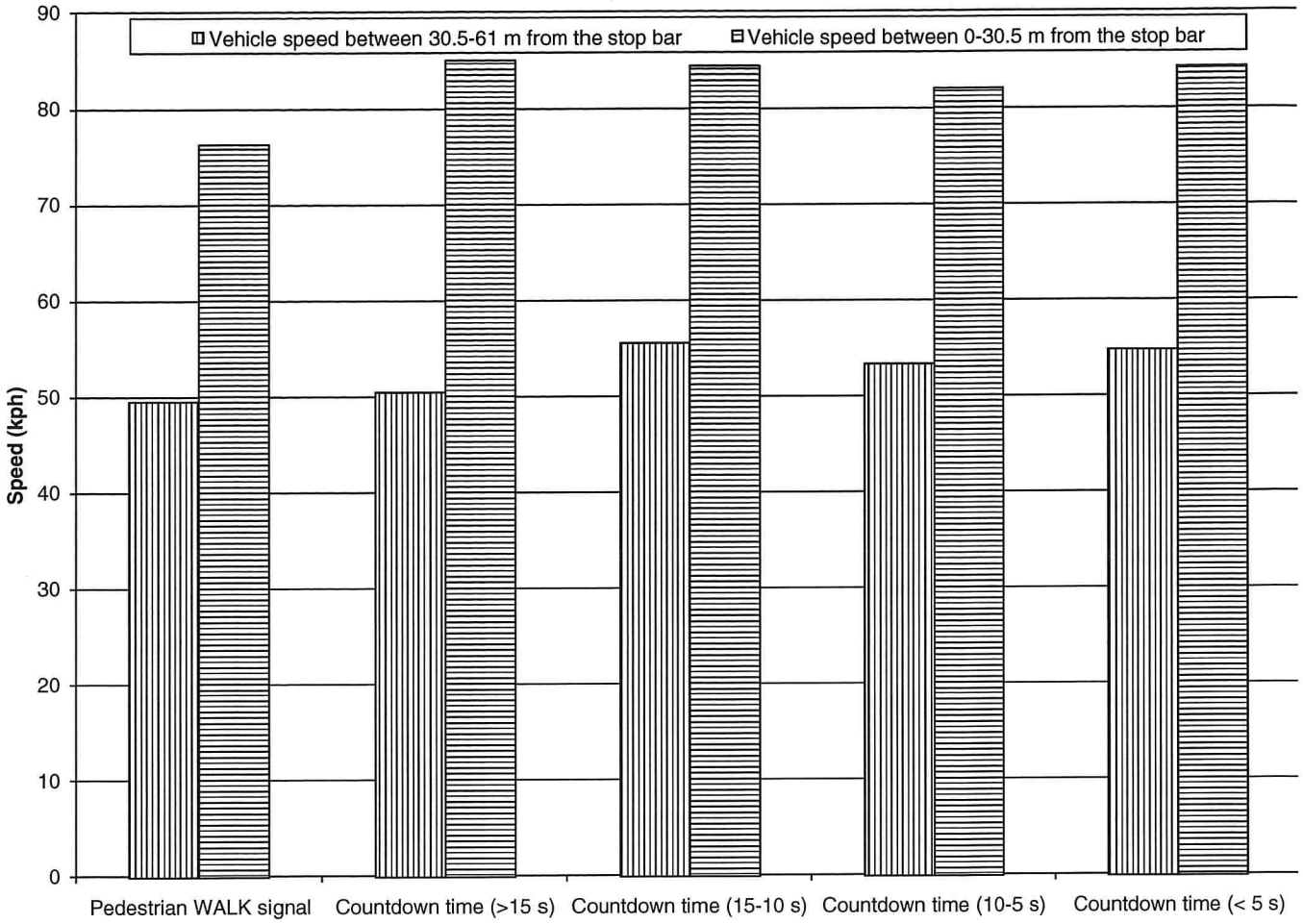


FIGURE 1 Mean speeds of vehicles for different pedestrian signal head indications at Green Valley Parkway and Warm Springs Road.

duration of the clearance interval, the all-red time, or other geometric features such as intersection width.

## SUMMARY AND CONCLUSIONS

This paper reports on an evaluation of the observed mean speeds of vehicles on intersection approaches with pedestrian countdown signals. Specifically, the speeds were observed on two segments immediately upstream of the stop bar. Observations of vehicle speeds were also made on the basis of five specific intervals identified on the basis of the indication on pedestrian countdown signals, during the "Walk" indication, and during each of the following countdown timer indications: greater than 15 s, 15 s to 10 s, 10 s to 5 s, and less than 5 s, respectively. The speeds were observed to be greater on the segment closer to the intersection than on the segment farther upstream of the intersection. The results clearly indicate that the speeds during the "Walk" display on the pedestrian signal are significantly lower than those during the countdown timer and flashing "Don't Walk" display on the signal head. However, results do not indicate that the speeds increase as the countdown clock winds down. All of these results were statistically significant at a 95% level of confidence, except one that had a 93% level of confidence. These findings validate other studies reported in the literature. Further, the results enhance the literature by adding information about speeds in spatial and temporal proximity to intersections with pedestrian countdown timers. As a part of the effort reported here, countdown timers were installed at locations where the study was conducted. Evaluations of comparable locations with flashing "Don't Walk" and in the presence of flashing "Don't Walk" with countdown timers merit further attention.

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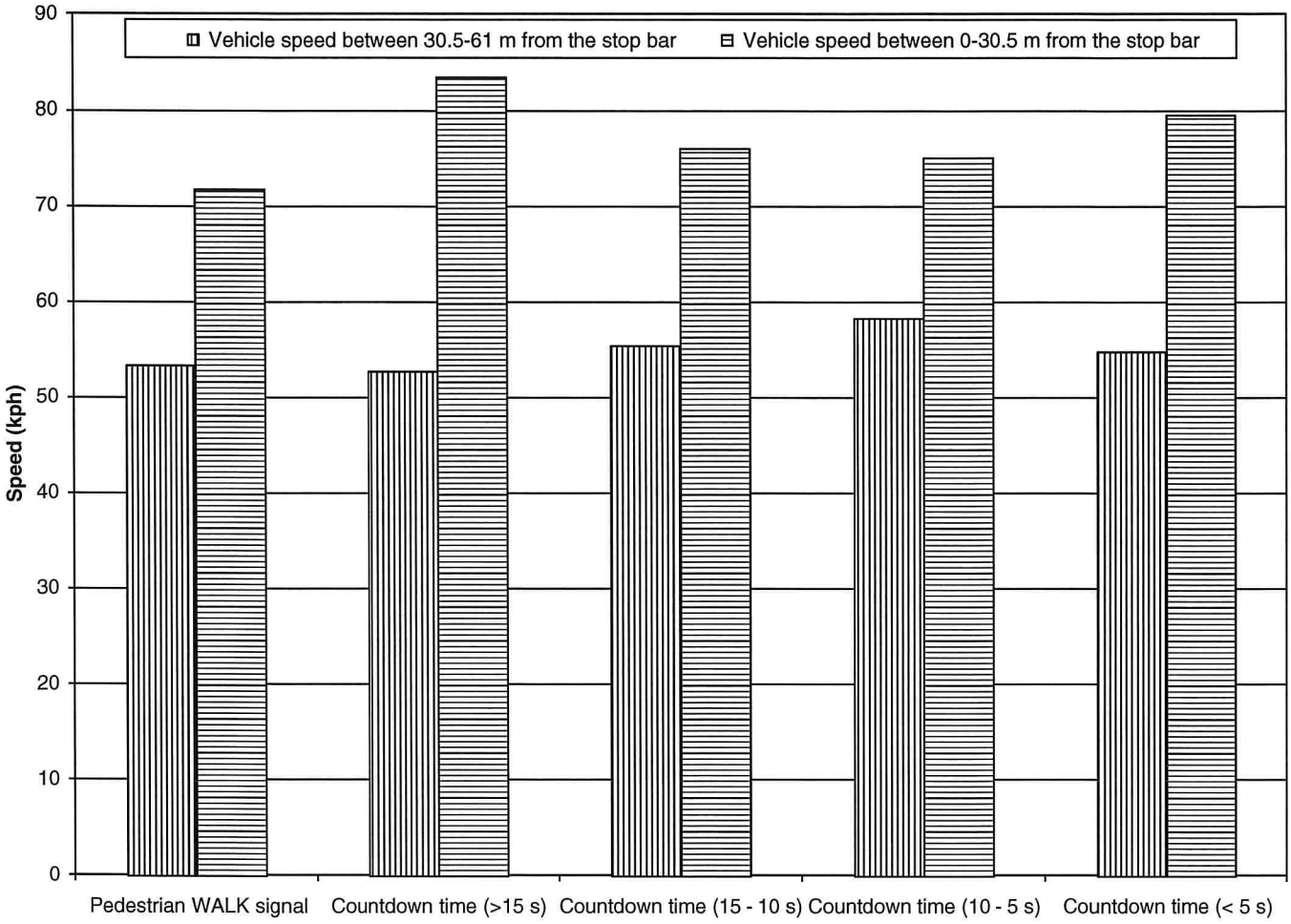


FIGURE 2 Mean speeds of vehicles for different pedestrian signal head indications at Sunset Road and Marks Street.

reviewers are thanked for their comments, which helped improve the clarity of the paper.

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*The Traffic Control Devices Committee peer-reviewed this paper.*

# Effects of Dynamically Activated Emergency Vehicle Warning Sign on Driver Behavior at Urban Intersections

Peter T. Savolainen, Tapan K. Datta, Indrajit Ghosh, and Timothy J. Gates

**Emergency vehicles are subjected to substantial risks when traveling through urban signalized intersections, particularly when the cross street traffic is provided with a green signal indication. Much of this risk is due to a lack of awareness of the approaching emergency vehicles on the part of the cross street traffic. To mitigate this problem, a dynamically activated emergency vehicle alert system was developed to provide an additional visual cue to motorists of an impending emergency vehicle's approach. The alert system consists of an LED case sign, mounted near the curb line adjacent to a traffic signal, which displays an emergency vehicle symbol when activated by a transmitter installed in the emergency vehicle as it approaches an intersection. A before-and-after evaluation of these warning signs was conducted at five test intersections in two suburban communities near Detroit, Michigan. The results showed that the alert system significantly improved driver awareness and behavior at all study locations as determined by comparing various measures of effectiveness. Overall, the proportion of drivers who yielded the right-of-way to an approaching emergency vehicle increased from 77.1% before installation of the warning signs to 96.6% after installation. Drivers also tended to yield sooner when the alert system was in use, and when violations of the emergency vehicle right-of-way did occur, they generally happened sooner.**

Emergency vehicles en route to incidents, including ambulances, fire trucks, and police cars, are encountered by the driving population on a daily basis. A recent questionnaire survey conducted in the city of Dearborn Heights, Michigan, revealed that more than 98% of respondents had encountered an emergency vehicle with lights and sirens activated on a public roadway (*1*). The incident response time of these emergency vehicles is often critical to saving lives and property, and any additional delay experienced en route to an incident can be a significant problem. In addition to delayed incident response, safety is also a serious public concern as evidenced by the 3,708 crashes that occurred in the state of Michigan from 2004 to 2008 involving in-service emergency vehicles (*2*). Emergency vehicle delay and safety are particular concerns at intersections, where more than one-third of these crashes occurred (*2*).

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A common cause of delay and emergency vehicle crashes, particularly at urban intersections, is the failure of the traveling public to yield the right-of-way to approaching emergency vehicles (*3–5*). Often, this failure is due to other drivers not being able to recognize the approaching emergency vehicles in time to react appropriately or failing to recognize these vehicles altogether. In the aforementioned Dearborn Heights survey, 82.9% of respondents indicated that they had noticed an in-service emergency vehicle, but were unable to react appropriately in the time available on at least one occasion (*1*).

To alleviate this problem, an emergency vehicle alert system (EVAS) was developed by a private contractor to provide a visual warning to road users, thus providing additional time for motorists to react appropriately to avoid traffic crashes or conflicts. The authors of this paper were engaged to perform a field evaluation to assess the real-world performance of this system, first in the city of Dearborn Heights and subsequently in the township of Grosse Ile, two suburban communities close to the city of Detroit, Michigan.

## BACKGROUND ON EMERGENCY VEHICLE WARNING TECHNOLOGIES

Late recognition or complete nonrecognition of emergency vehicles has long been recognized as a safety concern. Emerging in-vehicle distractions, such as radios and cell phones, may exacerbate this problem, especially at heavily traveled intersections. To minimize the crash risks at signalized intersections, emergency vehicle operators are trained to slow down or stop to reduce the potential for traffic crashes. In addition to driver training, rotating or oscillating emergency lights and sirens on the emergency vehicles function as a warning to road users, urging them to move aside to provide for safe passage of emergency vehicles. Despite these precautions, failure to recognize in-service emergency vehicles at intersections remains a problem.

Several technologies have been developed aimed at improving emergency vehicle operations and safety, the most widely used of which is known as emergency vehicle preemption (EVP), whereby the emergency vehicle's approach is given a green signal indication to provide safe passage through a signalized intersection during emergency runs. Past research has shown EVP to provide for savings of up to 45 s in intersection delay per run at high-volume intersections and average reductions of 14% to 23% in total emergency vehicle response time (*6*). In addition, substantial crash reductions have been experienced in communities in which EVP has been implemented on a wide scale (*6*). As of 2005 more than 30,964 signals were equipped with EVP technology in 375 separate jurisdictions across the country (*6*).

Several other traffic control devices have been developed in recent years to supplement EVP at signalized intersections. One such device, known as an EVP indicator, consists of a white floodlight mounted adjacent to the traffic signal, which is activated when an emergency vehicle approaches an intersection (*7*). These floodlights flash when an emergency vehicle is approaching an intersection from a side street and remain lit when an emergency vehicle is traveling on the same street. The EVP indicator has been installed in various communities in the United States, including Minneapolis and St. Paul, Minnesota (*7*).

Another emergency vehicle warning device has been installed at several intersections in Monrovia, California (*8*). This device, known as an emergency warning sign (EWS), uses a light-emitting diode (LED) display to alert motorists to the approach of an emergency vehicle (*9*). The sign displays a "Warning Emergency Vehicle" message, which is surrounded by eight ambulance silhouettes, one of which lights up to indicate the approach on which the emergency vehicle is currently traveling.

In addition to these devices, the U.S. Patent Office lists at least 32 patents that include the phrase "emergency vehicle warning system" (*10*). However, documentation could be found only to indicate the implementation of EVP and the two previously described devices. Although these two supplementary devices have been installed at various locations across the country, no evaluation of the operational effectiveness of either device was found in the existing research literature.

In general, determining the safety effects of emergency vehicle warning devices is difficult because of the relative infrequency of emergency vehicle crashes. In the absence of sufficient crash data, measuring changes in surrogate safety measures, such as the number of traffic conflicts, presents another means of assessing the efficacy of such warning devices (*6*). This paper presents the results of field studies that use various surrogate measures of safety to examine driver behavior during emergency vehicle runs that were conducted before and after the installation of the experimental emergency vehicle warning signs.

## DESCRIPTION OF EVAS

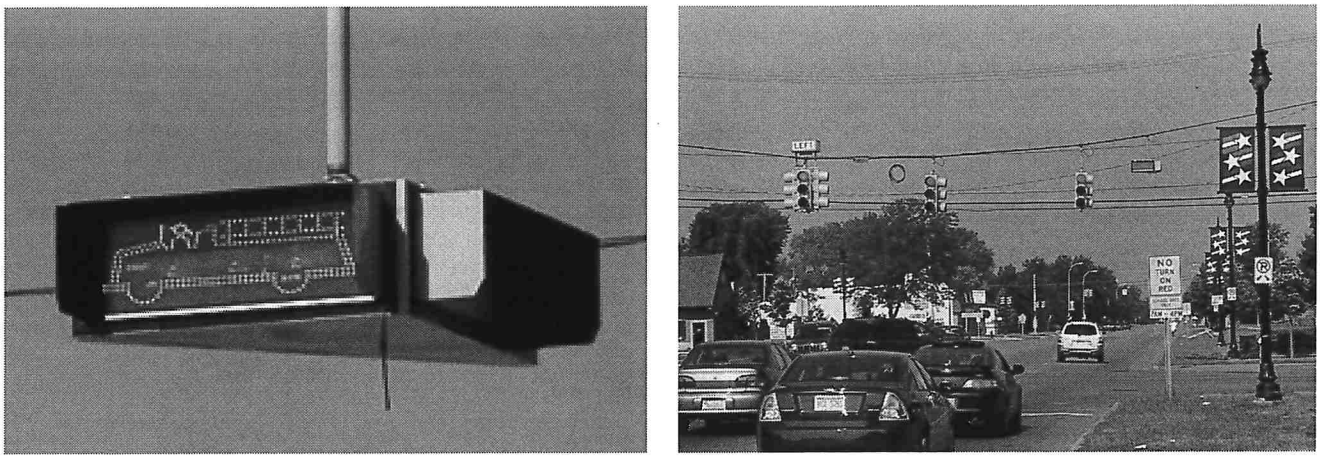
The alert system analyzed as a part of this study functions as a dynamic warning sign, displaying a fire truck image when activated [Emergency Vehicle Symbol W11-8 from the *Manual on Uniform Traffic Control Devices* (MUTCD)]. The warning sign, which is hung on the curbside adjacent to a traffic signal, consists of the case sign shown in Figure 1. The device uses 276 high-output amber LED lights mounted on a fixture that is 28.375 in. wide and 13.875 in. high. The system requires emergency vehicles to be equipped with a transmitter that can be activated on demand. The in-vehicle transmitters use a radio frequency of 418 MHz, which is a Department of Defense general control frequency for remote control devices. Transmissions use an encrypted 8-bit data stream to ensure the security of the system. This frequency band operates independently from that of signal preemption systems. Although it is theoretically possible to coordinate this alert system with signal preemption, this evaluation focuses solely on the effects of the warning signs that are under experimentation at five signalized intersections, none of which have any preemption capabilities. The intent of the device is to provide an additional visual warning to motorists; the device does not override the signal indication.

When an emergency vehicle begins to approach an intersection, the transmitter is activated and a signal is relayed to a receiver located on the light assembly. The transmitted signal can be detected from a distance of up to approximately 2,000 ft away. Until the signal is detected, the sign remains unlit with a dark face. When activated, the sign displays the fire truck image, providing a visual warning to motorists of the oncoming emergency vehicle (*11*). The display flashes once per second, as per MUTCD specifications, until the emergency vehicle has cleared the intersection (*11*). The receiver monitors the signal strength, and the warning sign ceases flashing when the signal strength drops below a specified threshold after a preset time interval. A schematic diagram illustrating the operation of the system is shown in Figure 2.

## STUDY DESIGN

To assess the effectiveness of the dynamic warning signs in improving traffic operations and safety, a before-and-after evaluation methodology was used. Five signalized intersections were selected for testing the system's effectiveness in improving intersection operations and safety during emergency vehicle runs. These test intersections included

1. Warren Avenue and Beech Daly Road in Dearborn Heights,
2. Warren Avenue and Fenton Street in Dearborn Heights,




(a) (b)

FIGURE 1 Photographs of emergency vehicle warning signs.

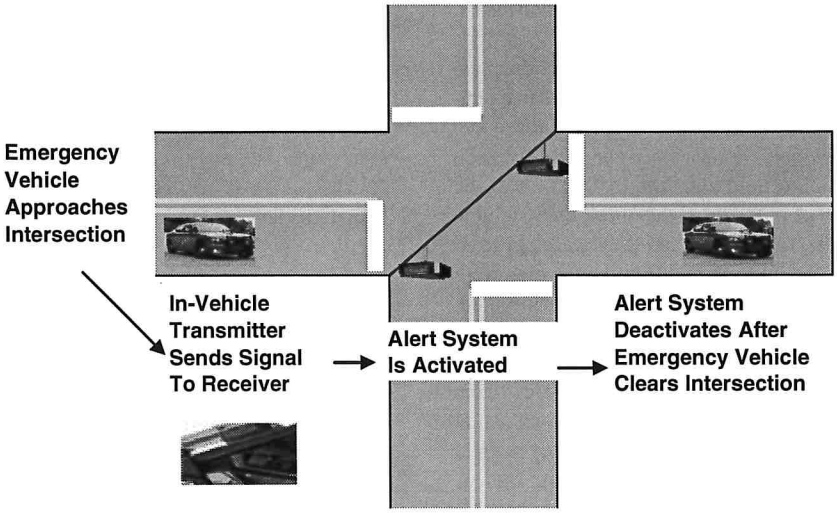

Emergency Vehicle Approaches Intersection
In-Vehicle Transmitter Sends Signal To Receiver
Alert System Is Activated
Alert System Deactivates After Emergency Vehicle Clears Intersection

FIGURE 2 Operation of EVAS.

3. Beech Daly Road and Hass Street in Dearborn Heights,
4. Grosse Ile Parkway and Meridian Road in Grosse Ile, and
5. Macomb Road and Meridian Road in Grosse Ile.

These particular intersections, shown in Figure 3, were selected as study locations in cooperation with participating emergency service departments as routes frequently traveled by emergency vehicles. Each of the study intersections currently operates fixed time traffic signals with a two-phase design, and none of the locations have preemption capabilities.

The Dearborn Heights Police and Fire Departments aided in developing the procedure for experimental testing, which was also

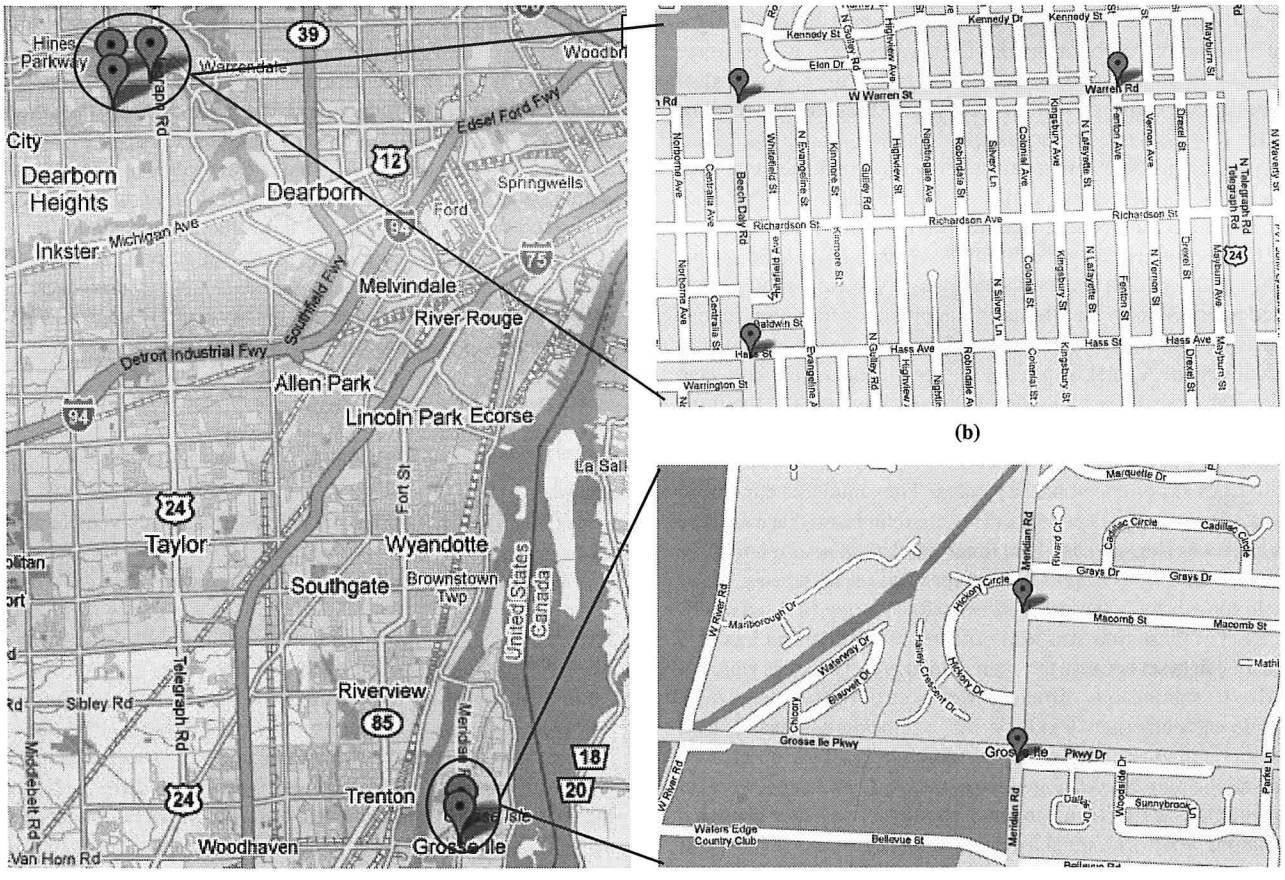



FIGURE 3 Map of (*a*) test site locations, (*b*) Dearborn Heights test sites, and (*c*) Grosse Ile test sites (*12*).

subsequently used by the Grosse Ile Police and Fire Departments. The experimental procedures were identical during the periods before and after installation of the alert systems, except for the presence of the devices themselves, and consisted of field tests to examine motorist compliance with the alert system at each of these five test intersections. Emergency vehicle runs were conducted at each of the study intersections before and after the installation of the warning signs. During each of these runs, the emergency vehicle drivers activated their lights and sirens when they were approximately 2,000 ft upstream of the intersection. Emergency runs were conducted in a similar manner after the warning signs were installed, with lights and sirens again activated 2,000 ft before the intersection, though the warning signs were also activated at this same point using the in-vehicle transmitters. All of the emergency vehicle drivers traveled through the test intersections as if they were on an actual emergency run. The runs were timed so that the emergency vehicle would arrive at the intersection shortly before the signal indication changed. Department standard operating procedures require emergency vehicle drivers to come to a complete stop when arriving at each intersection and proceed when conflicting traffic has yielded the right-of-way. Because emergency calls are random events, these simulated runs were necessary to ensure that an adequate sample of field data could be obtained to assess the effectiveness of the system's performance.

## DATA COLLECTION

Field data were collected before the installation of the warning signs at each of the three intersections in Dearborn Heights during June 2007 and at the two Grosse Ile sites during August 2008. The signs were installed in Dearborn Heights during July 2007 and in Grosse Ile during September 2008. Following installation, a period of 1 month passed before collection of the postinstallation data to allow time for drivers to become accustomed to the signs. To aid drivers in this task, a public awareness campaign was conducted that used radio, print, and television media. At the conclusion of this campaign, postinstallation data collection was performed during August 2007 in Dearborn Heights and during November 2008 in Grosse Ile.

On each day of data collection, emergency vehicle runs were conducted every 30 min on alternating approaches for a continuous 6-h period, resulting in a minimum of 12 runs during each day of data collection. A total of 6 h (i.e., 1 day) of data were collected before and after the installation of the warning signs at the three Dearborn Heights locations, and 12 h (i.e., 2 days) of data were collected at both of the Grosse Ile locations because of the lower traffic volumes experienced at these sites. The simulated runs were alternated between police, fire, and rescue vehicles so that each type of vehicle was used at least three times per day. In addition to these scheduled runs, several units were dispatched on actual emergency vehicle runs through the study intersections during the study periods. Each of these runs was also observed, recorded, and included in the final data set because the characteristics of these runs were not found to be significantly different from those of the simulated runs.

While each emergency vehicle run was being conducted, field observers simultaneously recorded motorist behavior on all four intersection approaches using video cameras. The cameras were set up on the curbside 150 ft upstream of the stop bar on each approach at locations where the signal heads, warning signs, and each lane of traffic were clearly visible. These video recordings captured all traffic entering the study intersections, beginning approximately 3 min before each emergency vehicle run and ending 3 min after each run. During recording, the field observers moved away from their cameras so as not to distract or influence the behaviors of approaching motorists. The video recordings were subsequently reviewed and analyzed to document the actions of all drivers during each emergency run. The resulting database was used to conduct a statistical analysis to determine the effects of the warning signs on traffic operations and safety.

## MEASURES OF EFFECTIVENESS

The emergency vehicle warning signs were designed to increase safety at intersections by warning road users of an approaching emergency vehicle. To determine the effects of the system on driver behavior, an examination of various measures of effectiveness (MOEs) was necessary. The most direct MOE for such a safety device is the frequency of traffic crashes experienced at each study location before and after installation of the warning signs. However, because of the random and infrequent nature of crashes involving emergency vehicles and the limited time period for which postinstallation crash data were available, the number of crashes did not provide an ideal MOE for this study. In fact, for the study intersections, only one emergency-vehicle-related crash was experienced in the 5 years before system installation and no such crashes have occurred after installation as of Nov. 14, 2009 (*2*).

Because of the infrequency of such crashes, surrogate measures of safety were necessary to allow for a timely assessment of the effects of the warning signs on driver behavior and intersection safety during the emergency runs. Surrogate safety measures are variables that are expected to be strongly associated with traffic crashes, but occurring more frequently such that sufficient data can be collected in a shorter period of time to provide for immediate feedback on safety performance (*13*). Traffic conflicts were initially examined as a potential surrogate measure of safety. Traffic conflicts are interactions in which one or more road users take evasive action, such as braking or weaving, to avoid a collision (*14, 15*). However, video from the initial field studies revealed traffic conflicts occurred relatively infrequently, so additional surrogate measures were selected for this evaluation to help discern changes in driver behavior as a result of the alert system. The specific MOEs for this study are as follows:

- Percentage of drivers yielding,
- Rate of violations per emergency run,
- Time between first yielding vehicle and emergency vehicle arrival,
- Time between latest violation and emergency vehicle arrival, and
- Clearance time for the emergency vehicle.

Before the statistical analyses were performed to determine the effects of the alert system on driver behavior, the data for each MOE were examined to determine whether the effect of the device varied on the basis of the type of emergency vehicle that was used during the emergency runs. The reason for this examination was twofold. First, if the data were not significantly different between the three types of emergency vehicles, it could all be combined into a single data set, increasing the power of the subsequent statistical tests. The other reason for this examination was to determine whether the device design was appropriate for use with all three

TABLE 1 Summary Statistics by Emergency Vehicle Type and Study Period

| | Emergency Vehicle Type and Study Period | | | | | |
|---|---|---|---|---|---|---|
| | EMS | | Fire | | Police | |
| Summary Statistic | Before | After | Before | After | Before | After |
| Number of emergency vehicle runs | 33 | 25 | 32 | 31 | 38 | 29 |
| Number of vehicles observed | 140 | 78 | 166 | 164 | 166 | 86 |
| Percentage of vehicles yielding | | | | | | |
| Overall | 75.7 | 97.4 | 77.7 | 97.6 | 77.7 | 94.2 |
| In right through lane | 89.7 | 100.0 | 85.7 | 97.9 | 85.7 | 100.0 |
| In left through lane | 74.3 | 100.0 | 61.4 | 97.8 | 65.4 | 81.0 |
| In left-turn lane | 16.7 | 80.0 | 70.6 | 95.2 | 73.9 | 100.0 |
| Before emergency vehicle in camera view | 35.0 | 74.4 | 28.9 | 79.3 | 42.2 | 73.3 |
| After crossing the intersection | 10.0 | 3.8 | 6.6 | 6.7 | 7.8 | 1.2 |
| Violations per emergency vehicle run | | | | | | |
| Total violations per run | 1.03 | 0.08 | 1.16 | 0.13 | 0.97 | 0.17 |
| Crossing violations per run | 0.55 | 0.00 | 0.91 | 0.10 | 0.71 | 0.14 |
| Right-turn violations per run | 0.03 | 0.00 | 0.09 | 0.00 | 0.03 | 0.03 |
| Left-turn violations per run | 0.45 | 0.08 | 0.16 | 0.03 | 0.24 | 0.00 |
| Time between first yielding vehicle and emergency vehicle arrival (s) | 5.53 | 10.86 | 8.10 | 13.67 | 5.39 | 10.72 |
| Time between latest violation and emergency vehicle arrival (s) | 4.76 | 12.52 | 4.10 | 8.46 | 4.89 | 6.13 |
| Clearance time for emergency vehicle (s) | 5.30 | 4.07 | 5.10 | 4.15 | 4.50 | 3.54 |

NOTE: EMS = emergency medical services.

types of emergency vehicles. The fire truck symbol used by the alert system is identical to the MUTCD-approved traffic sign (W11-8) used to indicate the presence of emergency vehicles (*11*). However, because ambulances and police cars will also use the system, it is important to ensure that the device functions effectively for all three types of vehicles. To make this determination, statistical tests were run to compare the performance of the warning device when each type of emergency vehicle traversed the intersection. Each of the previously discussed MOEs was compared between the emergency vehicle runs for each of the three vehicle types as shown in Table 1.

Two-sample Kolmogorov–Smirnov tests were first performed to determine whether the form of the underlying distributions for each MOE were significantly different between the runs conducted by the three types of emergency vehicles. For those MOEs with distributions that were not significantly different, standard analysis of variance (ANOVA) tests were conducted to compare the continuous variables, and the Marascuilo multiple comparison procedure was used to compare the MOEs that were expressed as percentages (*16*). For those MOEs that were continuous and whose distributions were found to be statistically different, the Kruskal–Wallis test was conducted. The Kruskal–Wallis test is a nonparametric alternative to traditional ANOVA that is more robust to nonnormality and resistant to outliers and nonconstant variance (*17*). The results of these statistical tests revealed that none of the MOE data for the three types of emergency vehicles were significantly different. This fact should be apparent from a visual examination of Table 1, which shows that the effects of the EVAS appear to be relatively consistent for each of the MOEs and each type of emergency vehicle.

Because the type of emergency vehicle used during a particular run did not appear to influence the effects of the dynamic warning signs, it may be concluded that the fire truck symbol is sufficient for alerting motorists of the approach of ambulances, fire trucks, or police cars. For analysis purposes, this also means that the data for each MOE can be combined across all emergency vehicle types, which increases the power of each of the statistical tests that were conducted. Consequently, the following sections present the results of the statistical tests that were conducted on the aggregated data set that included all types of emergency vehicles.

## STUDY RESULTS

Summary statistics for the aggregated data set are presented in Table 2, along with 95% confidence intervals for each MOE from the periods before and after installation of the warning signs. For each of the compliance-related MOEs expressed as a percentage, the *Z*-test of proportions was used to compare the compliance rates between the periods before and after installation of the dynamic warning signs. The Mann–Whitney *U*-test was used to examine changes in the violation rates between the periods before and after installation. The Mann–Whitney *U*-test (*16*) is a nonparametric alternative to the *t*-test and was necessary because the rate of violations per emergency vehicle run was not normally distributed as determined by a one-sample Kolmogorov–Smirnov test (*17*). Each of the remaining MOEs was appropriately characterized by a normal distribution, allowing for the use of an independent sample *t*-test. The differences in each selected MOE between the periods before and after installation of the warning signs were found to be statistically significantly at a 95% confidence level, with the exception of the rate of right-turn violations per emergency vehicle run. Although this MOE improved after installation of the warning signs, the number of right-turning vehicles observed at the study intersections was insufficient to determine a statistically significant difference. A further discussion of the results for each specific MOE follows.

TABLE 2 Aggregated Summary Statistics

| Summary Statistic | Study Period | |
|---|---|---|
| | Before | After |
| Number of emergency vehicle runs | 103 | 85 |
| Number of vehicles observed | 472 | 328 |
| Percentage of vehicles yielding | | |
| Overall | 77.1 ± 1.9 | 96.6 ± 1.0 |
| In right through lane | 86.9 ± 1.6 | 99.0 ± 0.5 |
| In left through lane | 66.4 ± 2.1 | 93.8 ± 1.3 |
| In left-turn lane | 55.2 ± 2.3 | 92.1 ± 1.5 |
| Before emergency vehicle in camera view | 35.4 ± 2.2 | 76.5 ± 2.3 |
| After crossing the intersection | 8.1 ± 1.4 | 4.6 ± 1.2 |
| Violations per emergency vehicle run | | |
| Total violations per run | 0.96 ± 0.32 | 0.13 ± 0.08 |
| Crossing violations per run | 0.66 ± 0.27 | 0.08 ± 0.06 |
| Right-turn violations per run | 0.05 ± 0.05 | 0.01 ± 0.02 |
| Left-turn violations per run | 0.25 ± 0.16 | 0.03 ± 0.05 |
| Time between first yielding vehicle and emergency vehicle arrival (s) | 6.29 ± 0.85 | 11.86 ± 1.81 |
| Time between latest violation and emergency vehicle arrival (s) | 4.63 ± 0.69 | 7.77 ± 2.05 |
| Clearance time for emergency vehicle (s) | 4.95 ± 0.37 | 3.92 ± 0.27 |

### Percentage of Drivers Yielding

In an optimal situation, all road users arriving at an intersection while an emergency vehicle is approaching will yield the right-of-way by pulling over to the right side of the road and allowing the emergency vehicle to pass. For this study, any driver who yielded before the time the emergency vehicle was within 150 ft of the stop bar was classified as compliant. The percentage of compliant (i.e., yielding) motorists increased from 77.1% to 96.6% after the alert systems were installed at the five test sites.

When performance on a lane-by-lane basis was examined, similar improvements were found. The compliance rates, both before and after installation of the warning signs, were found to be highest in the rightmost through lane and lowest in the left-turn lane. This is likely a function of the location of the alert system, which is installed near the curb line on the right side of the intersection. Left-turning drivers were less likely to notice the warning signs because of their location, a problem that was compounded by the diagonal span wire installation at each study intersection. However, even the vehicles in the left-turn lane had compliance rates of 92.1% after installation of the warning signs. The percentage of drivers who yielded before the emergency vehicle coming into the view of the cameras (150 ft upstream of the stop bar) also increased from 35.4% to 76.5%.

Vehicles were classified as yielding even if the driver proceeded through the intersection, noticed the approaching emergency vehicle while doing so, and subsequently yielded before the emergency vehicle reached the stop bar. Table 2 shows that before installation of the warning signs, 8.1% of drivers yielded after crossing the intersection. Afterward, this percentage was reduced to 4.6%.

### Violations

Any instance in which a driver passed completely through the intersection without yielding the right-of-way while the approaching emergency vehicle was within 150 ft of the stop bar upstream of the intersection was considered a violation. When provided with greater advance warning after the installation of the warning signs, drivers were able to identify approaching emergency vehicles more quickly. The results show a dramatic reduction in the rate of violations, which fell from an average of 1.05 per emergency vehicle run during the period before installation of the alert systems to only 0.13 per run afterward, a reduction of more than 87%. These improvements were consistent for each movement and travel lane.

### Time Between First Yield and Emergency Vehicle Arrival

In addition to the effect on the driver compliance rate, it was expected that the average time that elapsed between when the first vehicle yielded and the emergency vehicle arrived at the stop bar may also change after installation of the alert system. Before installation, the first vehicle to yield would typically do so approximately 6.29 s before the emergency vehicle's arrival. Following installation, vehicles began yielding up to 11.86 s before the emergency vehicle arrival. This result is likely due to drivers receiving notice of an approaching emergency vehicle sooner, because generally the system is activated before the emergency vehicle becomes visible to drivers near the intersection.

### Time Between Last Violation and Emergency Vehicle Arrival

Violations are possible with or without the dynamic warning signs. As the time interval between the latest violation and the arrival of the emergency vehicle increases, the potential for a crash decreases. Before the warning signs were installed, the latest violations occurred an average of 4.63 s before the emergency vehicle's arrival. Such a small time period does not provide much opportunity for either driver to react to a potentially hazardous situation. Following the installation of the alert system, this time period increased to an average of 7.12 s. Even though a small number of violations still occurred, they tended to occur much earlier and this may be due to some drivers noticing the emergency vehicle and ascertaining that they had sufficient time available to cross the intersection safely.

### Clearance Time for Emergency Vehicle

The faster an emergency vehicle is able to pass through (or clear) an intersection from stop bar to stop bar, the shorter the incident response time. After the dynamic warning signs were installed, emergency vehicle drivers were able to pass from stop bar to stop bar 1.03 s faster (4.95 s versus 3.92 s) than in the absence of the system. This result is due to the increased motorist compliance and, although emergency vehicle drivers were instructed to drive in a similar manner in both the before and after periods, a portion of this result may be due to the increased confidence on their part while proceeding through the intersection.

## CONCLUSIONS

The effectiveness of an experimental EVAS in improving driver awareness of an approaching emergency vehicle at urban signalized intersections was evaluated at five locations in Dearborn Heights and Grosse Ile, Michigan. Emergency vehicle runs (both simulated and actual) made during several days of data collection were recorded by

teams of field observers. The resulting video data were reviewed, compiled, and analyzed; and various measures of effectiveness were compared between the periods before and after installation of the warning signs. Overall, the warning signs were found to improve each measure of effectiveness and installation of the devices led to greater driver compliance and fewer violations of the emergency vehicles' right-of-way. Specific findings from the study are briefly summarized here:

- Compliance, in regard to the percentage of drivers yielding to the approaching emergency vehicle, increased from 77.1% to 96.6% after installation of the warning signs. This result was consistent for each travel lane.
- Drivers also tended to yield earlier after the alert systems were installed. The average time that elapsed between the first yielding vehicle and the emergency vehicle's arrival increased from 6.29 to 11.86 s. Before installation, 35.4% of drivers yielded before the emergency vehicle was within 150 ft of the stop bar. After installation, this percentage increased to 76.5%. Similarly, the percentage of vehicles that yielded after crossing the intersection decreased from 8.1% to 4.6%, indicating increased recognition by drivers.
- Before installation of the alert systems, an average of 1.05 violations occurred during each emergency vehicle run. After installation, this rate decreased to 0.13 violations per run. When violations did occur, generally they occurred much earlier, alleviating the potential hazard posed to both drivers. The time that elapsed between the last occurring violation and the emergency vehicle arrival increased from 4.63 to 7.77 s when the warning signs were in use.
- The time required for the emergency vehicle operators to clear the intersection from stop bar to stop bar decreased from 4.95 to 3.92 s after the devices were installed.
- Findings were consistent for each type of emergency vehicle, indicating that motorists are able to correctly associate MUTCD symbol W11-8 with police cars, ambulances, and fire trucks.

Collectively, the results of these before and after studies show that the dynamic emergency vehicle warning signs improve driver awareness of approaching emergency vehicles at urban intersections. This increased awareness has the potential to reduce the frequency of traffic conflicts and crashes involving emergency vehicles. These tests were conducted at intersections without signal preemption capabilities, and it is unclear how results may vary at locations in which preemption is used.

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# Dilemma Zone Driver Behavior as a Function of Vehicle Type, Time of Day, and Platooning

Timothy J. Gates and David A. Noyce

**A comprehensive investigation of the influence of vehicle type on various aspects of dilemma zone driver behavior, including brake response time, deceleration rate, and red light running occurrence, was performed at six urban or suburban signalized intersections in Wisconsin. Driver behavior data were obtained for 1,275 vehicles that were between 2.0 and 6.5 s upstream of the intersection at the onset of the yellow interval. Each vehicular observation was classified into one of five vehicle type categories: motorcycle, car, light truck (pickup, SUV, van, minivan), single-unit truck (single-unit heavy truck, delivery truck, recreation vehicle, bus), and tractor trailer (multiunit heavy truck). Each observation was also classified by time of day and whether the subject vehicle was part of a platoon. Vehicle type had a statistically significant effect on deceleration rate and red light running occurrence but did not have an effect on brake response time. Deceleration rates were highest for cars and light trucks; single-unit trucks and tractor trailers showed the lowest deceleration rates. Tractor trailers were 3.6 times more likely and single-unit trucks were 2.5 times more likely to commit red light running compared with passenger vehicles. The rates of red light running for cars and light trucks were not substantially different from each other. Time of day (peak versus off-peak) had a statistically significant effect on both deceleration rate and occurrence of red light running. Deceleration rates were significantly higher during off-peak times. Red light running was 1.3 times more likely to occur during peak periods compared with off-peak periods. Platooning had no effect on any of the measures of effectiveness.**

When a traffic signal changes from a green indication to a yellow indication, approaching drivers often face the dilemma of whether to stop or proceed through the intersection. The term "dilemma zone" or, more appropriately, "indecision zone" is often used to describe this occurrence (*1*). A detailed literature review by Bonneson et al. found that typically the dilemma zone exists between 2.5 and 5.5 s upstream of the intersection at the start of the yellow interval (*2*), which is often assumed to represent the threshold between which approximately 10% and 90% of drivers stop in response to the yellow (*3*). Throughout this paper, the term "dilemma zone" will be used to represent the "indecision zone" situation as it is the more familiar term.

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Previous research during the past several decades has investigated various characteristics of driver behavior in the dilemma zone (*1–13*), including brake response times, deceleration rates, the probability of stopping versus going through, and red light running. However, the published literature shows relatively little investigation into the differences in dilemma zone driver behavior as a function of the type of vehicle. The only relevant analyses that were found in the literature were limited to comparison of passenger vehicles versus heavy trucks. Zegeer and Deen found that heavy trucks committed red light running at more than twice the rate of passenger vehicles (*3*). A similar result was reported by Bonneson et al., who found heavy trucks to be more than twice as likely to commit red light running compared with passenger vehicles (*13*). Schultz performed an extensive literature review on nonemergency deceleration rates for passenger vehicles versus heavy trucks (*14*). Nonemergency deceleration rates for passenger vehicles fell within a range of 7 ft/s$^2$ and 12 ft/s$^2$. Truck deceleration rates were found to be slightly lower than those of passenger cars, particularly for trucks not equipped with antilock brakes, which showed deceleration rates for nonemergency stopping between 5 ft/s$^2$ and 9 ft/s$^2$. Trucks with antilock brakes decelerated between 10 and 11.6 ft/s$^2$—similar to the middle range of values observed for passenger vehicles.

Previous work by Gates et al. provided a comprehensive investigation of dilemma zone driver behavior, including differences between passenger vehicles and heavy vehicles (i.e., buses, recreational vehicles, single-unit trucks, and semitrailers) (*12*). Heavy vehicles were found to be less likely to stop when presented with a yellow indication and were more likely to commit red light running. Heavy vehicles in the dilemma zone were also found to use a lower deceleration rate when stopping and a shorter brake response time. Although the Gates et al. work provided a comprehensive analysis of dilemma zone driver behavior, because of a limited sample size for heavy vehicles, it did not provide a complete analysis of the effects of vehicle type, beyond that of passenger vehicles versus heavy vehicles. Recent expansion of the data set used by Gates et al. allowed for a comprehensive investigation of vehicle type to be performed, the results of which are described here.

## GOAL AND OBJECTIVES

The primary goal of this study was to provide a comprehensive investigation of the influence of vehicle type on various aspects of dilemma zone driver behavior. The specific behavioral characteristics for dilemma zone drivers that were of interest included

- Brake response time,
- Deceleration rate, and
- Occurrence of red light running.

Therefore, the study objectives were to determine the differences in brake response time and deceleration rate and to predict the likelihood of a red light running event as a function of vehicle type. The effects of other factors, including time of day (proxied by peak versus off-peak times) and whether the subject vehicle was part of a platoon of vehicles (defined by a headway or tailway of 2.0 s or less), were also investigated.

## METHODOLOGY

### Field Procedures

A field study was performed at six signalized intersection approaches in greater Madison, Wisconsin. The characteristics of each study site are included in Table 1. Data were collected using a video camera mounted on a 20-ft-tall steel pole that was temporarily attached to a roadside signpost that was between 400 and 800 ft upstream of the intersection. The camera was aimed toward the intersection so that the rear of vehicles could be viewed while approaching the intersection on the subject approach. During installation it was also ensured that the camera obtained a clear view of the entire intersection including the traffic signal indication. Data were collected only during dry conditions and daylight hours to prevent damage or vandalism to the video camera. Full details of the field study procedures are described in a recent publication by Gates et al. (*12*).

### Video Review and Data Extraction

Approximately 43 h of video were obtained during data collection activities. The researchers reviewed the video data and recorded several attributes related to the behavior of all dilemma zone vehicles that were either the last vehicle to go through or the first vehicle to stop in each lane for each signal cycle. A vehicle was considered in the dilemma zone if the front of the vehicle was between 2.0 and 6.5 s upstream of the intersection at the start of the yellow, determined on the basis of the subject vehicle's approach speed and distance from the intersection stop line. This time range represented a dilemma zone that was slightly larger than that cited in a detailed literature review by Bonneson et al. (*2*), which was deemed necessary to capture dilemma zone behavior for all vehicle types. Red light running vehicles that were more than 6.5 s upstream at the start of the yellow were also included in the sample. Similarly, stopping vehicles that were closer than 2.0 s from the intersection at the start of yellow were also included in the data set. Only one red light running event that was more than 6.5 s away from the intersection was observed. The following information was obtained from the video for each subject vehicle included in the sample:

- Time to traverse the initial 50 ft (for speed computation);
- Position and time at the onset of yellow;
- Position and time after the onset of yellow when the brake light became illuminated (stopping vehicles only);
- Time required for the vehicle to stop after the brake lights became illuminated (stopping vehicles only);
- Time elapsed from the onset of yellow until entry into the intersection (go through vehicles only);
- Action of the vehicle:
  - Stopped,
  - Went through but entered the intersection before the end of the yellow, or
  - Went through but entered the intersection after the end of the yellow (i.e., red light running);
- Time of day:
  - Peak (7–9 a.m. or 4–6 p.m.) or
  - Off-peak (all other observation times);
- Platooning:
  - Platooned (headway or tailway less than or equal to 2 s) or
  - Not platooned (both headway and tailway greater than 2 s);
- Vehicle type:
  - Motorcycle,
  - Passenger car,
  - SUV,
  - Pickup,
  - Minivan,
  - Van,
  - Bus,
  - Recreational vehicle (RV),

TABLE 1 Site Characteristic

| Characteristic | Johnson at Park | Verona at Raymond | Verona at McKee | John Nolen at Lakeside | Fish Hatchery at Caddis | East Washington at Baldwin |
|---|---|---|---|---|---|---|
| Subject approach | Eastbound | Northbound | Northbound | Southbound | Southbound | Westbound |
| Speed limit (mph) | 25 | 40 | 50 | 45 | 40 | 35 |
| Approach grade (%) | 0.0 | –0.3 | 1.1 | –0.7 | 1.9 | –0.7 |
| Cycle length (s) | 110 (peak)<br>80 (off-peak) | Variable | Variable | Variable | Variable | 80 |
| Yellow duration (s) | 3.5 | 4.5 | 5.0 | 4.0 | 4.0 | 3.5 |
| All-red time (s) | 3.0 | 1.75 | 2.0 | 1.5 | 1.0 | 1.0 |
| Intersection width (ft) | 90 | 90 | 125 | 80 | 90 | 70 |
| Signal actuation | Pretimed | Fully actuated | Fully actuated | Fully actuated | Fully actuated | Pretimed |
| Signal coordination[a] | C | U | U | U | U | C |
| Area type | Urban | Suburban | Suburban | Suburban | Suburban | Urban |

[a]C = coordinated and U = uncoordinated.

– Single-unit truck, or
– Tractor trailer.

Vehicular observations were excluded from the analysis for any of the following reasons:

- Turned right or left at the intersection,
- Braked before the onset of yellow (stopping vehicles only), and
- Presence of a queue on the subject approach.

For purposes of this study, red light running events were defined as cases in which the front of the vehicle did not reach the intersection stop line by the onset of the red indication.

## Data Reduction and Coding

The raw time and positioning information obtained for each subject vehicle were used to compute approach speeds, estimated travel time to the intersection stop line at the onset of yellow, brake response times, and deceleration rates for each vehicle. Approach speeds (ft/s) were calculated using the vehicle's time to traverse the initial 50 ft of the intersection approach. The estimated travel time to the intersection at the onset of yellow was calculated by dividing the subject vehicle's distance from the stop line at the onset of yellow by its approach speed. Brake response times were computed as the difference between the time at start of yellow and the time when the brake lights became illuminated. The occurrence of driver "coasting" (i.e., removing foot from accelerator and not immediately applying the brake) could not be quantified from review of the video. The average deceleration rate was computed for each vehicle on the basis of the approach speed and braking time. Braking time was computed as the difference between the time that the brake lights became illuminated and the time that the vehicle had stopped. The following formula was used to compute the average deceleration rate:

$$\text{decel rate}\left(\text{ft/s}^2\right) = \frac{\text{approach speed}}{\text{braking time}} \quad (1)$$

Vehicle type was initially classified into several specific categories for each type of vehicle. However, small sample sizes for several of the heavy-vehicle categories coupled with the impracticality of discerning between certain types of vehicles, such as some SUVs, minivans, and station wagons, led the researchers to consolidate the vehicle type variable into five categories for analysis, which included

- Motorcycle,
- Car,
- Light truck (pickups, SUVs, vans, minivans),
- Single-unit truck (all single-unit heavy trucks, delivery trucks, RVs, buses), and
- Tractor trailer (all multiunit heavy trucks).

## Data Classifications and Distributions

The data set included records for 1,275 dilemma zone vehicles. A cross tabulation of the vehicular observations separated by action and vehicle type is shown in Table 2.

Table 2 shows that 52.9% of the observed dilemma zone vehicles were passenger cars, and 38.5% of the observations were categorized as light trucks. Single-unit trucks and tractor trailers made up 5.0% and 3.1% of the observations, respectively, and motorcycles made up only 0.5% of the observations. For comparison purposes, recent data for vehicle miles traveled (vmt) on urban non-Interstates were obtained from the FHWA website. FHWA data show that cars accounted for 57.8% of the vmt; light trucks, 37.3%; single-unit trucks and buses, 2.5%; tractor trailers, 2.0%; and motorcycles, 0.4% (*15*). Thus, compared with FHWA proportions, cars were slightly underrepresented in the data set, and heavy vehicles were slightly overrepresented. Stopping vehicles and vehicles entering the intersection before the start of red accounted for 47.2% and 46.4% of the observations, respectively; red light running vehicles accounted for the remaining 6.4% of the observations. Red light running occurred at a rate of approximately 1.9 events per hour, which was lower than the 4.9 events per hour observed by Bonneson et al. at 10 Texas intersections (*13*). Red light running was considered only for through vehicles for the study described here.

For 29 of the 602 first-to-stop vehicles, brake response times and deceleration rates could not be discerned from the videotapes because of obstruction by other vehicles, reflections, or other visibility issues. Furthermore, because only one stopping motorcycle was observed, motorcycle was removed as a vehicle type category in the brake response and deceleration rate analysis. Thus, deceleration rates and

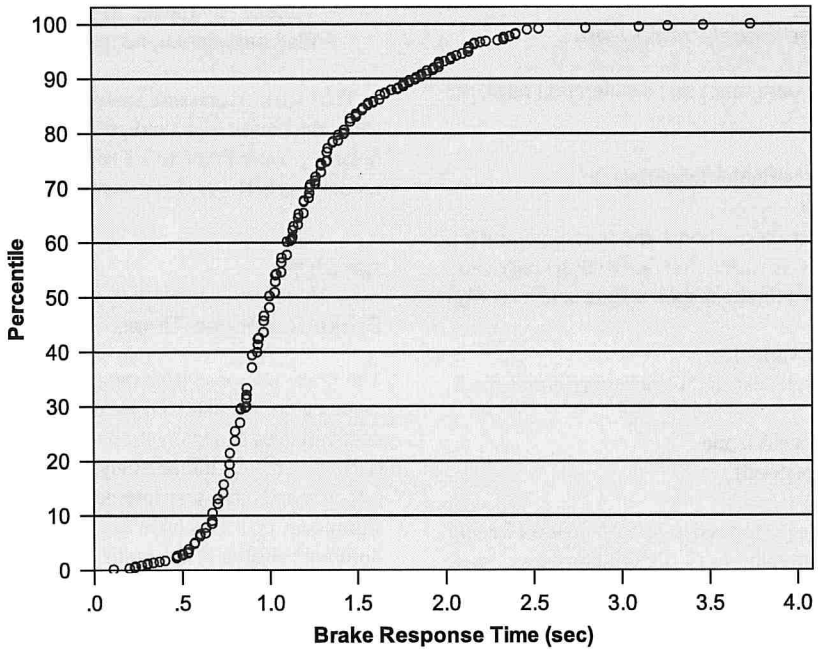
TABLE 2 Vehicular Action by Vehicle Type

| | Action | | | | |
|---|---|---|---|---|---|
| Vehicle Type | Stopped | Went Through—Entered Before Red | Went Through—Entered After Red (RLR) | Count | Percentage |
| Car | 332 | 298 | 44 | 674 | 52.9 |
| Light truck | 236 | 234 | 21 | 491 | 38.5 |
| Single-unit truck | 25 | 30 | 9 | 64 | 5.0 |
| Tractor trailer | 8 | 24 | 8 | 40 | 3.1 |
| Motorcycle | 1 | 5 | 0 | 6 | 0.5 |
| Total count | 602 | 591 | 82 | 1,275 | |
| Total percent | 47.2 | 46.4 | 6.4 | | 100.0 |

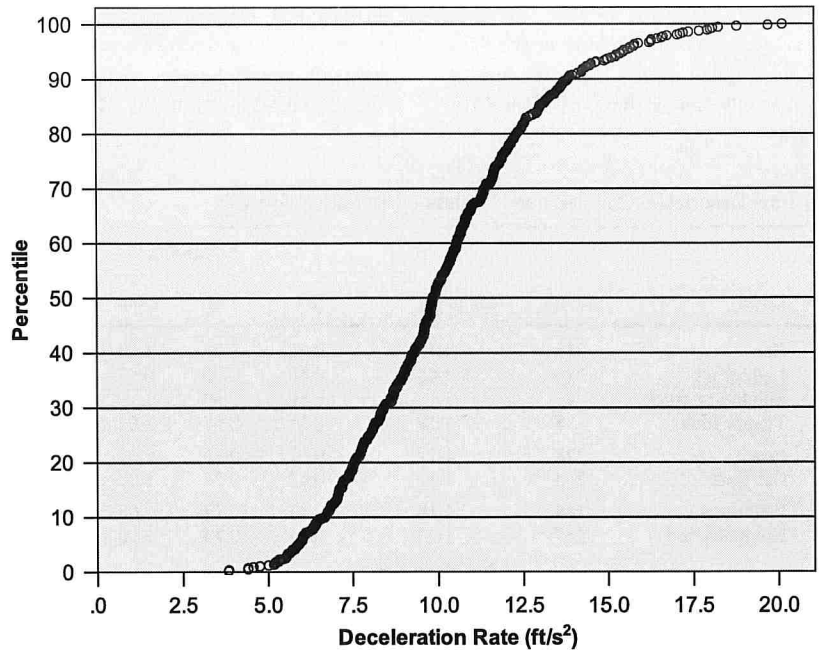
NOTE: RLR = red light running.

brake response times for 572 first-to-stop vehicles were included in the analysis. Figure 1 displays the cumulative distributions of (*a*) brake response times and (*b*) deceleration rates for the aggregated data.

The brake response times ranged from 0.11 to 3.74 s, with a mean and standard deviation of 1.13 and 0.48 s, respectively. The 15th, 50th, and 85th percentile brake response times were 0.73, 1.00, and 1.57 s, respectively. Deceleration rates ranged from 3.83 to 20.12 ft/s$^2$, with a mean and standard deviation of 10.13 and 2.86 ft/s$^2$, respectively. The 15th, 50th, and 85th percentile deceleration rates were 7.10, 9.87, and 13.01 ft/s$^2$, respectively. The mean values and cumulative distributions for the aggregated brake response time and deceleration rate data were similar to those reported in previous research (*5–7*). Furthermore, the median brake response time (1.00 s) and deceleration rate (9.87 ft/s$^2$) were in agreement with the respective values recommended by ITE for timing of the yellow interval on the basis of elimination of the dilemma zone (*16*).




(a)




(b)

FIGURE 1 Cumulative distributions for (*a*) brake response time and (*b*) deceleration rate.

### Analytical Procedures

Three primary analyses were performed using the appropriate statistical procedures. The dependent variables for these analyses included the following (statistical procedure shown in parenthesis):

- Brake response time for first-to-stop vehicles [univariate analysis of variance with covariates (ANOVA)],
- Deceleration rate for first-to-stop vehicles (ANOVA), and
- Likelihood of a red light running event (logistic regression).

#### *Brake Response Time and Deceleration Rate*

For the analyses of brake response times and deceleration rates, the independent variables included

- Continuous independent variables (covariates):
  - Approach speed (mph),
  - Estimated travel time to intersection at the start of yellow (s),
  - Brake response times (s) (deceleration rate analysis only), and
  - Deceleration rates (ft/s$^2$) (brake response time analysis only) and
- Categorical independent variables:
  - Vehicle type (motorcycle, car, light truck, single-unit truck, tractor trailer),
  - Time of day (peak, off peak), and
  - Platoon (platoon, nonplatoon).

The ANOVA analyses were performed in SPSS Version 17 using the general linear model command (*17*). Full-factorial analyses were performed, which included the main-factor effects in addition to two-way and three-way interactions of the main effects.

#### *Red Light Running Likelihood*

Stepwise binary logistic regression was used to determine the likelihood of red light running events. Logistic regression is a technique used to predict the probability of an outcome on the basis of values of a set of predictor variables (continuous or categorical) and is similar to linear regression except that the response variable is categorical rather than numeric. For the analysis of red light running events the independent variables entered into stepwise model included

- Continuous independent variables:
  - Approach speed (mph) and
  - Estimated travel time to intersection at the start of yellow (s) and
- Categorical independent variables:
  - Vehicle type (motorcycle, car, light truck, single-unit truck, tractor trailer),
  - Time of day (peak, off peak), and
  - Platoon (platoon, nonplatoon).

The logistic regression analysis was performed in SPSS Version 17 using the binary logistic regression command (*17*). The confidence level for a predictor to be removed from the backward stepwise model was 0.10 (0.05 for reentry into the model).

## RESULTS

### Brake Response Times

The brake response time data were analyzed using the analysis of variance statistical technique to determine the effect of the independent variables and interactions of the independent variables on brake response times. Although they were included in the ANOVA model, two-way and three-way interactions were not found to be statistically significant and thus have been excluded from further discussion. Summary results of the statistical analysis along with the relevant descriptive statistics are shown in Table 3.

The full factorial ANOVA model including all interactions (not shown in Table 3) was statistically significant (at 95% confidence) and showed an adequate $R^2$ value (.528), indicating that the variability in brake response time is partially explained by the factors included in the model. Three of the six main independent variables entered into the stepwise model were found to significantly affect brake response, although none of the categorical variables, including vehicle type, were found to have a significant effect. Each of the covariates, approach

TABLE 3 Brake Response Time Descriptive Statistics and Results of Statistical Analysis

| | | | | | Percentiles (s) | | | ANOVA | |
|---|---|---|---|---|---|---|---|---|---|
| Factor | Level | Count | Mean (s) | SD | 15th | 50th | 85th | *F*-Statistic | *p*-Value |
| Vehicle type | Car | 315 | 1.17 | 0.50 | 0.77 | 1.03 | 1.64 | 1.729 | .160 |
| | Light truck | 226 | 1.08 | 0.46 | 0.70 | 0.97 | 1.47 | | |
| | Single-unit truck | 23 | 1.17 | 0.50 | 0.59 | 1.10 | 1.65 | | |
| | Tractor trailer | 8 | 1.18 | 0.61 | 0.57 | 1.02 | 2.13 | | |
| Time of day | Peak | 228 | 1.17 | 0.51 | 0.77 | 1.07 | 1.61 | 1.697 | .193 |
| | Off-peak | 344 | 1.11 | 0.47 | 0.72 | 1.00 | 1.54 | | |
| Platoon | Platooned | 185 | 1.14 | 0.47 | 0.77 | 1.03 | 1.60 | .918 | .338 |
| | Not platooned | 387 | 1.13 | 0.49 | 0.73 | 1.00 | 1.57 | | |
| Speed | | | Not applicable | | | | | 412.4 | .000[a] |
| Travel time to intersection | | | Not applicable | | | | | 528.6 | .000[a] |
| Deceleration rate | | | Not applicable | | | | | 424.6 | .000[a] |
| Full model[b] | All data | 572 | 1.13 | 0.48 | 0.73 | 1.00 | 1.57 | 41.527 | .000[a] |

[a]Indicates that the factor was statistically significant at a 95% confidence level.
[b]$R^2$ = .528. Two-way and three-way factor interactions were included in the analysis, but were excluded from the table, as they were not statistically significant.

speed, travel time to the intersection, and deceleration rate, were found to have similar effects on deceleration rate in regard to magnitude, as indicated by the *F*-statistic. Further investigation of the effects of the covariates indicated that brake response time decreased as approach speed increased (i.e., faster drivers reacted more quickly), increased as travel time from the intersection increased (i.e., drivers reacted more slowly when farther from the intersection), and increased as the deceleration rate increased (i.e., drivers reacted more slowly if a greater deceleration rate was subsequently used).

## Deceleration Rates

The deceleration rate data were also analyzed using the analysis of variance statistical technique to determine the effect of the independent variables and interactions of the independent variables on deceleration rates. Although they were included in the ANOVA model, two-way and three-way interactions were not found to be statistically significant and thus have been excluded from further discussion. The summary results of the statistical analysis along with the relevant descriptive statistics are shown in Table 4.

The full factorial ANOVA model including all interactions (not shown in Table 4) was statistically significant (at 95% confidence) and showed a relatively high $R^2$ value (.815), indicating that most of the variability in deceleration rate is explained by the factors included in the model. Five of the six main independent variables entered into the step-by-step model were found to significantly affect deceleration rate. Whether the subject vehicle was platooned or not was the only factor that was not statistically significant. Of the statistically significant variables, approach speed and travel time to the intersection were found to have the strongest effect on deceleration rate, as indicated by the *F*-statistic. Brake response time was also found to have a significant effect on deceleration rate, although this effect was not as strong as speed and travel time. Further investigation of the effects of the covariates indicated that deceleration rate increased as approach speed increased (i.e., faster drivers used greater deceleration), decreased as travel time from the intersection increased (i.e., drivers used lower deceleration when farther from the intersection), and increased as the brake response time increased (i.e., slower-reacting drivers subsequently used greater deceleration rates). Figure 2 displays two-way scatterplots for (*a*) deceleration rate and approach speed and (*b*) deceleration rate and travel time. The correlations of dilemma zone deceleration rates with travel time, approach speed, and brake response time were similar to those found by Chang et al. (*5*).

### *Effect of Vehicle Type*

Vehicle type was found to have a statistically significant effect on deceleration rate, although the magnitude of the effect was much smaller than that observed for speed, travel time, and brake response time. The cumulative distribution of deceleration rates by vehicle type is shown in Figure 3. The mean deceleration rates for each vehicle type along with the 95% confidence interval for the means are shown in Figure 4.

Mean deceleration rates were highest for the car and light truck categories, although light truck deceleration rates were slightly, but not statistically significantly, higher than those for cars. The mean deceleration rate for single-unit trucks was significantly lower than that for both cars and light trucks. The mean deceleration rate for tractor trailers was similar to that of single-unit trucks and considerably lower than that for cars and light trucks, but was not statistically different from that for any of the other vehicle types, a result in large part of the small sample size.

### *Effect of Time of Day*

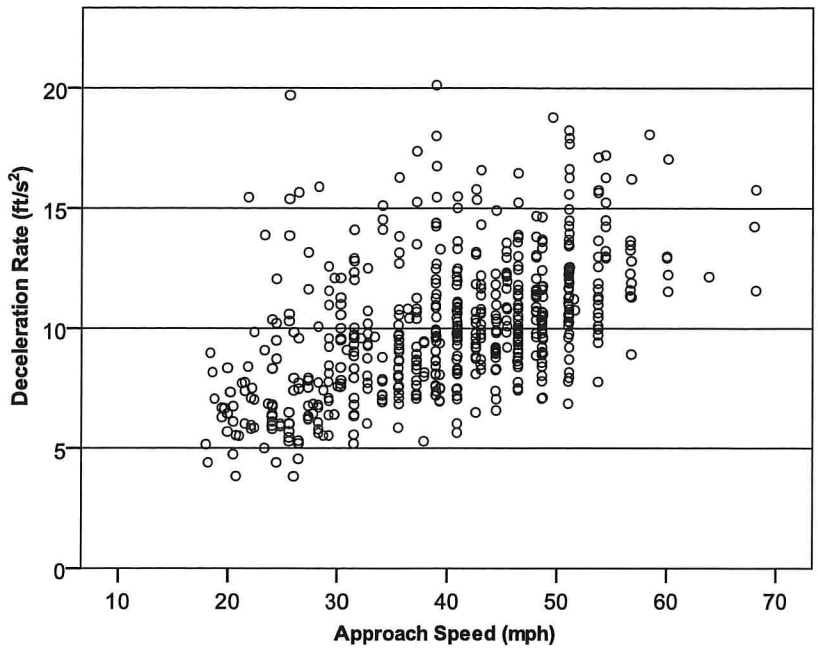
Time of day was found to have a statistically significant effect on deceleration rate, although the magnitude of the effect was much smaller than that observed for speed, travel time, and brake response time. The mean deceleration rates for peak versus off-peak times along with the 95% confidence interval for the means are shown in Figure 5.

Mean deceleration rates were significantly higher during off-peak times. This was likely a result of dilemma zone drivers being less inclined to stop during peak times, particularly if a relatively high deceleration rate was necessary to stop.

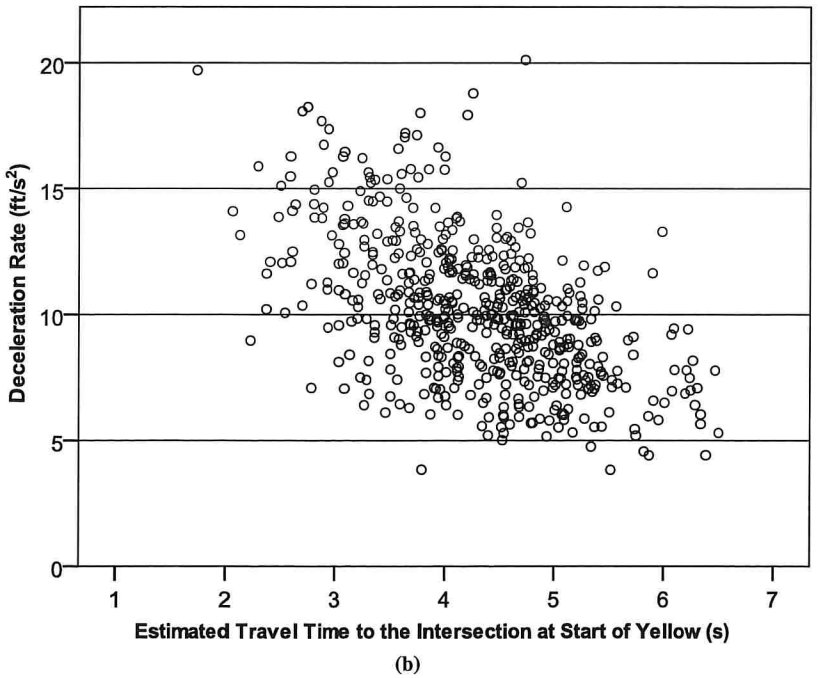
TABLE 4 Deceleration Rate Descriptive Statistics and Results of Statistical Analysis

| | | | | | Percentiles (ft/s²) | | | ANOVA | |
|---|---|---|---|---|---|---|---|---|---|
| Factor | Level | Count | Mean (ft/s²) | SD (ft/s²) | 15th | 50th | 85th | *F*-Statistic | *p*-Value |
| Vehicle type | Car | 315 | 10.09 | 2.95 | 7.00 | 9.78 | 13.12 | 3.163 | .024[a] |
| | Light truck | 226 | 10.42 | 2.72 | 7.46 | 10.33 | 13.29 | | |
| | Single-unit truck | 23 | 8.18 | 2.12 | 5.91 | 7.73 | 10.95 | | |
| | Tractor trailer | 8 | 8.59 | 1.95 | 6.37 | 8.54 | 11.30 | | |
| Time of day | Peak | 228 | 9.87 | 3.03 | 6.46 | 9.65 | 13.20 | 5.104 | .024[a] |
| | Off-peak | 344 | 10.30 | 2.72 | 7.49 | 10.01 | 12.96 | | |
| Platoon | Platooned | 185 | 9.86 | 2.86 | 6.86 | 9.68 | 12.58 | 2.246 | .135 |
| | Not platooned | 387 | 10.25 | 2.85 | 7.23 | 10.00 | 13.31 | | |
| Speed | | | Not applicable | | | | | 1,207.9 | .000[a] |
| Travel time to intersection | | | Not applicable | | | | | 1,419.5 | .000[a] |
| Brake response time | | | Not applicable | | | | | 424.6 | .000[a] |
| Full model[b] | All data | 572 | 10.13 | 2.86 | 7.10 | 9.87 | 13.01 | 162.8 | .000[a] |

[a]Indicates that the factor was statistically significant at a 95% confidence level.
[b]$R^2$ = .815. Two-way and three-way factor interactions were included in the analysis, but were excluded from the table, as they were not statistically significant.




(a)




(b)

FIGURE 2 Scatterplots for (*a*) deceleration rate versus approach speed and (*b*) deceleration rate versus travel time.

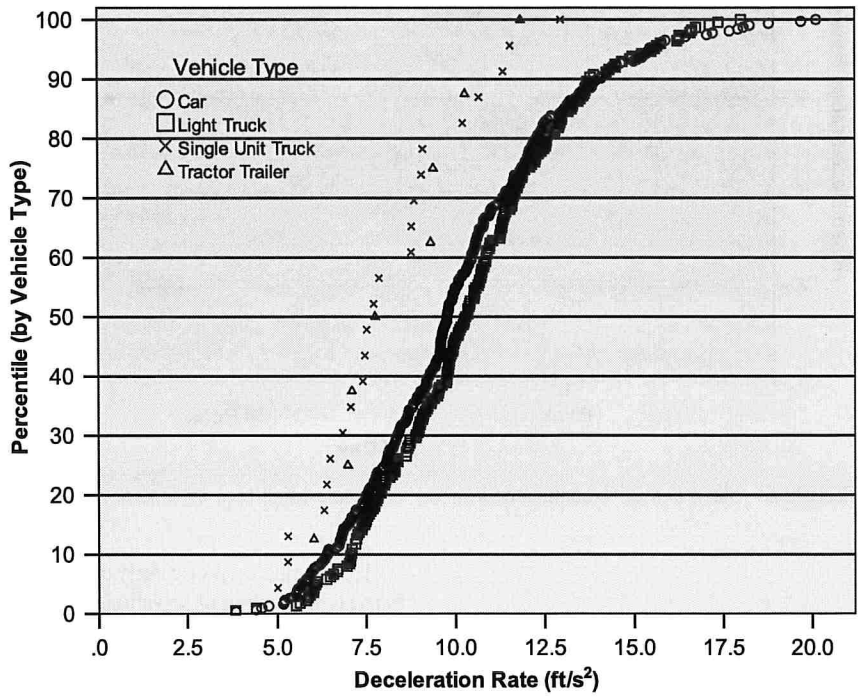



FIGURE 3 Cumulative distribution for deceleration rate by vehicle type.

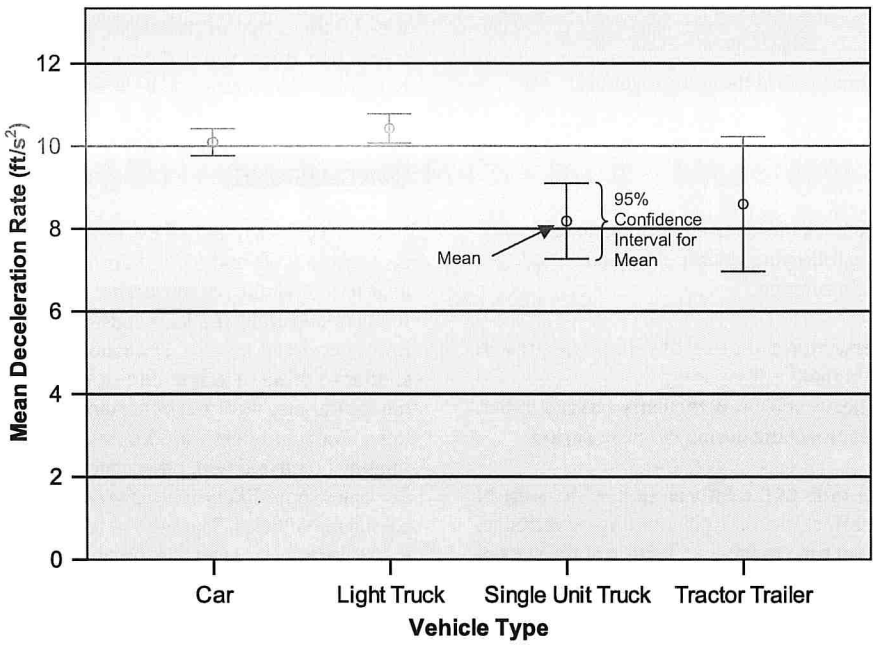



FIGURE 4 Mean and 95% confidence interval for deceleration rate by vehicle type.

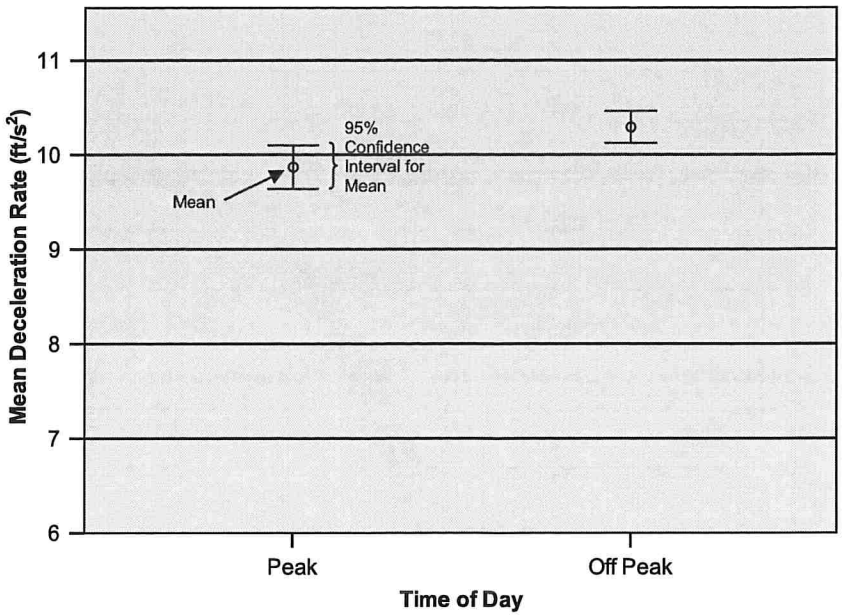



FIGURE 5 Mean and 95% confidence interval for deceleration rate by time of day.

### Red Light Running

Of the 1,275 vehicles in the data set, 82 red light running events were observed, accounting for 6.4% of the vehicles. The logistic regression analysis showed that red light running events were difficult to predict on the basis of the potential predictor variables used here, largely because red light running events often occur as a result of the attitude or inattention level of the individual driver. Nevertheless, the following variables were found to significantly affect red light running occurrence:

- Travel time to the intersection at the start of yellow,
- Approach speed,
- Vehicle type, and
- Time of day.

The direction of the parameter estimates from the logistic regression analysis indicated that the following conditions contributed to a higher likelihood of red light running:

- Travel time to the intersection at the start of yellow was greater.
- Approach speed was higher.
- Subject vehicle was a heavy vehicle, particularly a tractor trailer.
- Subject vehicle was approaching during the peak period.

As expected, the travel time to the intersection at the start of yellow had the strongest effect on red light running occurrence. The median estimated travel time to the intersection at the start of yellow for the red light runners was 4.45 s, compared with 3.68 s for all other vehicles. Also as expected, faster drivers were more likely to commit red light running. The mean approach speed for red light running vehicles was 45.81 mph compared with 42.28 mph for all other vehicles. The time of day was also significant in that drivers were more likely to commit red light running during peak times compared with off-peak times. Red light running accounted for 7.6% of the peak-period observations and 5.7% of the off-peak observations.

Heavy vehicles, particularly tractor trailers, were overrepresented in red light running observations. Red light running was committed by 20.0% of tractor trailers and 14.1% of single-unit trucks (16.3% of all heavy vehicles combined), whereas only 6.5% of cars and 4.3% of light trucks committed red light running (5.6% of all passenger vehicles combined). In regard to relative rates of occurrence, tractor trailers were 3.6 times more likely and single-unit trucks were 2.5 times more likely to commit red light running compared with passenger vehicles. The red light running rate of occurrence for heavy vehicles was 2.9 times that of passenger vehicles, which is consistent with research by Bonneson et al. that found heavy vehicles to be 2.3 times more likely to commit red light running (*13*).

## CONCLUSIONS

Vehicle type was found to have a statistically significant effect on both deceleration rate and red light running occurrence but did not have an effect on brake response time. Deceleration rates were highest for cars and light trucks; single-unit trucks showed significantly lower deceleration rates. Deceleration rates for tractor trailers were similar to those of single-unit trucks. Heavy vehicles, particularly tractor trailers, were overrepresented in red light running observations. Tractor trailers were 3.6 times more likely to commit red light running compared with passenger vehicles. Single-unit trucks were 2.5 times more likely to commit red light running compared with passenger vehicles. The rates of red light running for cars and light trucks were not substantially different from each other. The differences in stopping behavior between passenger vehicles and heavy vehicles were expected because heavy vehicle operators are less likely to stop because of several reasons, including (*a*) heavy vehicles cannot stop as rapidly as passenger vehicles, (*b*) heavy vehicle operators typically have higher delay-related costs, and (*c*) heavy vehicle operators may avoid using high deceleration rates during nonemergencies to prevent shifting of cargo.

The time of day (i.e., peak versus off peak) had a statistically significant effect on both deceleration rate and occurrence of red light

running. Deceleration rates were significantly higher during off-peak times. Red light running was 1.3 times more likely to occur during peak periods compared with off-peak periods. Both of these results were expected because drivers are less inclined to stop during peak periods as a result of any or all of the following: (*a*) greater time pressure, (*b*) greater levels of delay when stopped at signalized intersections, (*c*) greater uncertainty of the actions of trailing drivers, and (*d*) a perceived reduction in the threat of being cited for committing red light running. Brake response times were not significantly affected by time of day. Whether the subject vehicle was part of a platoon or not had no significant effect on deceleration rates, brake response times, or red light running occurrence.

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*The Traffic Control Devices Committee peer-reviewed this paper.*

# Relative Visibility of Internally Versus Externally Illuminated On-Premise Signs

Philip M. Garvey, Stephen J. Damin, Martin T. Pietrucha, and Damian Deptuch

**Poorly visible on-premise commercial signs have been associated with reduced safety because drivers trying to locate and make sense of these signs may drive slower than the rest of traffic and perform erratic, last-second maneuvers. One of the main reasons for reduced sign visibility is poor sign lighting. In addressing this issue, research has demonstrated that internally illuminated on-premise signs have 40% to 60% greater visibility than externally illuminated signs on a controlled test track. Even so, an ever-increasing number of jurisdictions are implementing sign ordinances that prohibit the use of internally illuminated on-premise signs, mainly for aesthetic reasons. The objective of this research was to expand on the earlier test track research by evaluating the relative visibility of internally and externally illuminated signs on open roads in the real world. The results of this research clearly demonstrate the superiority of internally illuminated signs across a wide variety of driving conditions, sign offsets, sizes, shapes, colors, external lighting designs, and quality.**

A substantial body of research has established that illuminated on-premise commercial signs play a critical role in providing wayfinding information and more general guidance and orientation to motorists at night (*1*). By supplying drivers with important navigational cues at critical decision points where traffic signs and even GPS systems leave off, illuminated on-premise signs play a positive role in traffic safety and comfort. This is true especially after dark, when visual performance and environmental cues decline. However, for any sign to be effective, it must be legible from a distance at which a driver can process the sign's content and respond to its information safely.

Many factors contribute to sign legibility, with one of the most important being luminance (*2*). Documented research shows most on-premise sign lighting to be within reasonable bounds (*3*). At night, however, dimly illuminated on-premise signs can be very difficult or impossible to read at appropriate distances, overloading a driver's perceptual and cognitive resources, leading to erratic driving maneuvers such as inappropriate rates of deceleration and untimely lane changes, and overly bright signs can cause annoying and potentially disabling light pollution (*4*).

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Signs are illuminated using two basic strategies, the selection of which can have a dramatic effect on their brightness and the uniformity of light distribution across the sign face. Although a sign might be lit using various devices, including neon tubes, LEDs, and either incandescent or halogen lamps, all signs fall into one of two major lighting categories: internally illuminated or externally illuminated. These terms were defined by the Illuminating Engineering Society of North America as follows (*4*):

1. Externally illuminated signs: signs on which a fixed message is illuminated by an external source of light and
2. Internally illuminated signs: signs in which the source of light is enclosed within the sign and the message becomes understandable because of the varying transmittance of the sign face.

Although most highway signs are illuminated externally via the interaction between retroreflective materials and vehicle headlamps, on most roadways in commercial, industrial, and office building environments, the method of on-premise sign illumination is internal.

Past research sponsored by the United States Sign Council Foundation demonstrated that internally illuminated on-premise signs have 40% to 60% greater legibility than externally illuminated signs in a controlled test track environment (*5*). Even so, an ever-increasing number of jurisdictions are implementing sign ordinances that prohibit the use of internally illuminated on-premise signs, mainly for aesthetic reasons. The objective of the present research was to expand on the earlier test track work by evaluating the relative visibility of internally and externally illuminated signs on open roads in the real world.

## METHODOLOGY

The study was an older and younger driver, gender-balanced, human factors evaluation of the nighttime sign visibility of commercial on-premise sign lighting design. The general methodology was an open field, or "real world," study in which a representative sample of the driving population was asked to find and read internally and externally illuminated signs on actual storefront properties while operating a vehicle on in-use roadways.

### Variables

The critical independent variable was on-premise sign lighting design (internal versus external illumination). Additional variables included driver age, gender, visual acuity, familiarity, weather, and

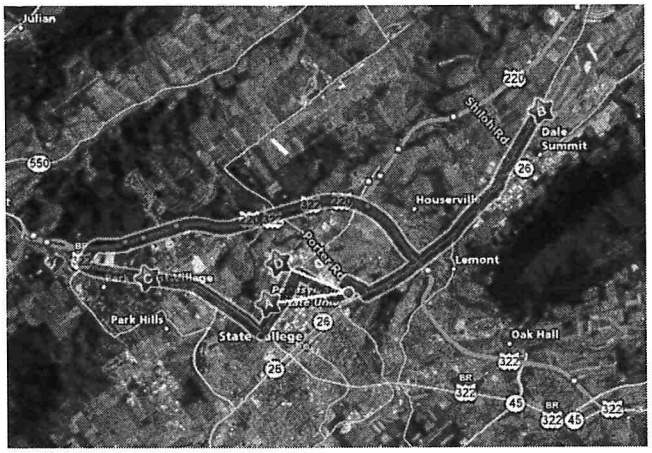


FIGURE 1 Test route.

driving speed. The dependent variable (or measure of effectiveness) was a real-world combination of detection and legibility distance used effectively in earlier research (*6*).

### Location

The six signs were located on sections of US-26 and US-322 Business in State College, Pennsylvania (Figure 1; A and D are the start and end points, B and C are the two farthest signs). At four of the six sign locations, US-322 and US-26 are major arterials with five-lane (one turn lane), two-way cross sections with curbing. At two of the sign locations on US-26 (Summit and Fine Line), the cross section drops to three lanes (one turn lane), two way. The posted speed limit on the approach to the Animal Medical Hospital and Viet Thai Restaurant signs was 35 mph (56 km/h); the posted speed for the approaches to the remaining four signs was 45 mph (72 km/h).

### Externally Illuminated Signs

Six existing in-use, externally illuminated signs were selected for this study (Figure 2). In the previous research evaluating the relative legibility of internally and externally illuminated signs, Garvey and his colleagues optimized external sign illumination with the use of clean, new flood lamps aimed with precision at the signs, ensuring a high level of uniform illumination throughout the evaluation (*5*). The externally illuminated signs selected for the current study better reflect what drivers are exposed to in the real world in that they varied in lighting quality and brightness level from poor to excellent (Figure 3; Table 1).

### Internally Illuminated Signs

A set of internally illuminated signs identical to the six existing, externally illuminated signs in copy (e.g., message, letter height, font, and spacing), sign shape, color, contrast orientation, and size were designed and fabricated by volunteer United States Sign Council members (Figures 4 and 5). The nighttime lighting levels and design were based on sign industry standards that in earlier research had been found to be optimal for these signs (*7*).

### Photometric Characteristics

Experimenters documented the luminance of the internally and existing externally illuminated signs using a Minolta LS-110 luminance meter (Table 2) and techniques successfully developed in earlier research (*3, 7*). Five of the signs fell within recommended levels to avoid glare effects for rural, suburban, and urban environmental lighting (i.e., Environmental Zones E2, E3, and E4) under both internal and external illumination, with the sixth (Summit Chiropractic Clinic) accommodating Lighting Zones E3 and E4 when internally illuminated and Zones E2, E3, and E4 when externally illuminated (*3*).

### Subjects

Eighty subjects participated in the research. Forty viewed the internally illuminated signs and 40 viewed the externally illuminated signs. Half of the subjects that viewed each lighting condition were female and half were male. All subjects had valid U.S. driver's licenses. The subjects were selected to represent the U.S. driving population in age (Table 3). The subjects' binocular, static distance visual acuity was measured using a Sloan letter chart displayed on a Good-Light Company light box. The mean visual acuity for the subjects who viewed the internally illuminated signs was 20/20 (6/6), and the mean visual acuity for those who saw the externally illuminated signs was 20/19 (6/5.7).

## PROCEDURE

All 80 subjects drove a 2004 Dodge Stratus sedan along a half-hour route through State College, Pennsylvania, commercial districts at night. The subjects were accompanied by an experimenter in the passenger seat and one in the rear seat. The vehicle was instrumented with a Nu-Metrics distance measuring instrument (DMI) to record sign visibility distances.

The subjects were given simple route directions to follow and were instructed to drive "as they normally would" while emphasizing safety and maintaining the posted speed limits. To simulate the common experience drivers have when they know what business establishment they are looking for, but do not know its location, the subjects were told the name of the establishment and were asked to read the sign aloud as soon as they could. The moment the subjects read the signs correctly, the experimenter in the passenger seat pressed a button on the DMI. The button was pressed a second time when the vehicle was alongside the signs. The DMI calculated the distance between the two button presses and recorded the result as the visibility distance for that condition.

The internally illuminated signs were placed in front of and blocking the externally illuminated signs while the first half of the subjects participated. The internally illuminated signs were then removed and the second half of the subjects viewed the externally illuminated signs using the same procedures.

## ANALYSES AND RESULTS

### Gender and Age

Forty males and 40 females participated in the study. On average, the males read the signs at 233 ft (71.02 m) and the females at 225 ft (68.58 m). An analysis of variance (ANOVA) was conducted, and not

FIGURE 2 Externally illuminated signs, daytime.

FIGURE 3 Externally illuminated signs, nighttime.

TABLE 1 Description of External Lighting Equipment and Placement

| Sign | Distance of Lamps to Sign (ft) | Number of Lamps | Type of Bulb | Bulb Wattage |
|---|---|---|---|---|
| Marrara's | 5 | 2 | Halogen | 300 |
| Summit | 4 | 1 | Mercury | 100 |
| Fine Line | 3 | 1 | Metal halide | 50 |
| Glantz, Johnson | 6.5 | 1 | Fluorescent reflector lamp | 26 |
| Animal Medical Hospital | 8 | 1 | Halogen | 100 |
| Viet Thai | 6 | 1 | Halogen | Unmarked bulb between 100 and 200 |

NOTE: 1 ft = 0.3048 m.

surprisingly, the difference was not statistically significant ($F = 0.28$; $p = .60$). Another ANOVA showed that there was also no statistically significant age group effect ($F = 1.58$; $p = .20$), with the youngest age group reading the signs at, on average, 213 ft (64.92 m), the two middle groups at 249 and 236 ft (75.9 and 71.93 m), and the oldest group at 202 ft (61.6 m).

### Visual Acuity, Familiarity, Weather

Three separate ANOVAs were conducted on the variables visual acuity, familiarity, and weather. There were no statistically significant effects as a function of subject static visual acuity ($F = 1.72$; $p = .16$). Although visual acuity is often found to be a good predictor of sign legibility, this was not the case for the small range in visual acuity that the subjects exhibited combined with the complex task of finding and reading signs in the real world while driving in live traffic at night.

A portion of the subjects were from the local State College area and knew the location of some of the business establishments where the six test signs were mounted. The statistical analysis showed slight improvement (254 versus 221 ft) (77.42 versus 67.36 m) in the distance at which the signs were found and read as a function of sign familiarity ($F = 4.55$; $p = .04$). Some of the data were collected during light rain or light snow or when the roads were wet. An analysis of the data showed that this did not significantly affect sign detection and reading distances ($F = 0.59$; $p = .63$).

### Speed

Two separate statistical analyses were conducted on the two lighting conditions to determine whether there was a significant correlation between speed and sign reading distance in this study. The hypothesis was that when drivers have difficulty reading a sign, they will slow down, which presents potential traffic safety concerns. Although the $R^2$ values were small (hovering around .10), they were statistically significant ($t = 4.93$, $p < .001$ for externally illuminated signs; and $t = 4.83$, $p < .001$ for internally illuminated signs), revealing that the drivers in this study did indeed drive more slowly around less visible signs (Figure 6).

### Sign

Sign visibility differed across the six signs (Figure 7, the numbers in the bars indicate the number of subjects who drove past the sign without ever seeing it). The average reading distances of the six signs varied because of uncontrolled differences, including location, including placement on the left (Summit) or right side of the road (all others); lateral and vertical sign offset; roadway characteristics (e.g., number of lanes of traffic and curvature); and characteristics of the signs themselves, perhaps, most importantly, size, color, and shape.

The statistical Glimmix procedure was used to determine which of the signs were significantly affected by lighting design. The result was that all of the signs performed statistically significantly better with internal illumination. The biggest improvement was with the Animal Medical Hospital sign, which was read on average 2.36 times farther away with internal illumination. This was a 196-ft (59.74-m) mean difference, giving drivers almost 4 extra seconds at 35 mph (56 km/h). Furthermore, this sign was missed by two drivers when it was externally illuminated. Even the most modest increase (Marrara's Dry Cleaner, which was maximally externally illuminated with two 300-watt halogen lamps) resulted in almost 1.35 extra seconds of driver reaction time.

### Internal Versus External Illumination

The Tukey–Kramer statistical test was used to evaluate the combined visibility of all six signs tested. The test showed a statistically significant improvement in sign visibility when the sign was internally illuminated ($t = -10.19$; $p < .001$). The internally illuminated signs were visible on average 68% farther away than the externally illuminated signs (291 versus 173 ft) (88.1 versus 52.73 m). This is a 118-ft (35.97-m) difference, which at 35 mph (56 km/h) means that drivers have an additional 2.3 s to read and react to the externally illuminated signs [1.8 s at 45 mph (72 km/h)].

## CONCLUSIONS

The primary objective of this research project was to conduct a one-to-one comparison between internally and externally illuminated on-premise signs on open roadways, using real drivers and in-use signs. To fairly evaluate the differences in nighttime visibility between signs that are internally illuminated and signs that are externally illuminated, the signs must be identical in all aspects other than lighting design. This was accomplished by fabricating exact internally illuminated replicas of the existing externally illuminated signs and placing them in front of the existing signs, so that not just the signs but the locations and offsets (and therefore the visual surround and roadway characteristics) were identical.

FIGURE 4 Internally illuminated signs, daytime.

FIGURE 5 Internally illuminated signs, nighttime.

TABLE 2 Nighttime Sign Luminance (cd/m²)

| Sign | Color | Internal Illumination (cd/m²) | External Illumination (cd/m²) |
|---|---|---|---|
| Marrara's | Red | 15 | 13 |
| | Yellow | 150 | 60 |
| Summit | White | 700 | 15 |
| Fine Line | Green | 60 | 1.0 |
| | Gold (inlay letters) | 80 | 10 |
| Glantz, Johnson | Green | 30 | 0.5 |
| | White | 180 | 2.9 |
| Animal Medical Hospital | Dark blue | 20 | 0.25 |
| | Light blue | 130 | 1.25 |
| | Gold (inlay letters) | 130 | 1.5 |
| Viet Thai | Brown | 40 | 0.5 |
| | Red | 110 | 2.5 |
| | Green | 187 | 1.25 |
| | Pink | 260 | 5.0 |
| | Yellow | 325 | 8.0 |

TABLE 3 Subject Age Group and Visual Acuity Data

| Age Group | Percent of U.S. Driving Population (*8*) | Number of Subjects (half viewed internally illuminated sign and half externally) | Mean Visual Acuity |
|---|---|---|---|
| 18–29 | 20.1 | $n = 16$ | 20/19 |
| 30–44 | 28.4 | $n = 24$ | 20/18 |
| 45–59 | 28.4 | $n = 24$ | 20/20 |
| 60+ | 21.2 | $n = 16$ | 20/23 |

NOTE: Visual acuity of 20/20 = 6/6.

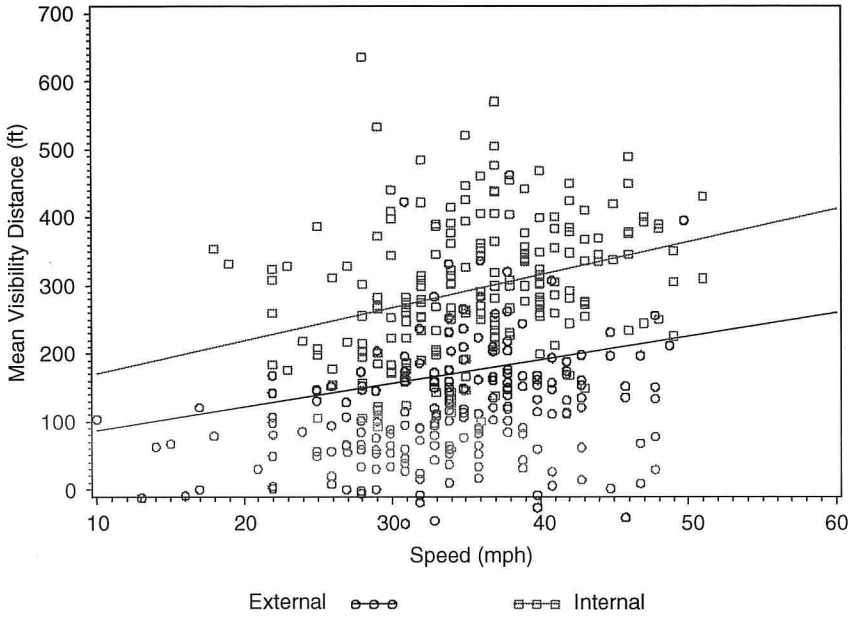


FIGURE 6 Scatterplot of sign visibility distance in feet by speed in miles per hour.

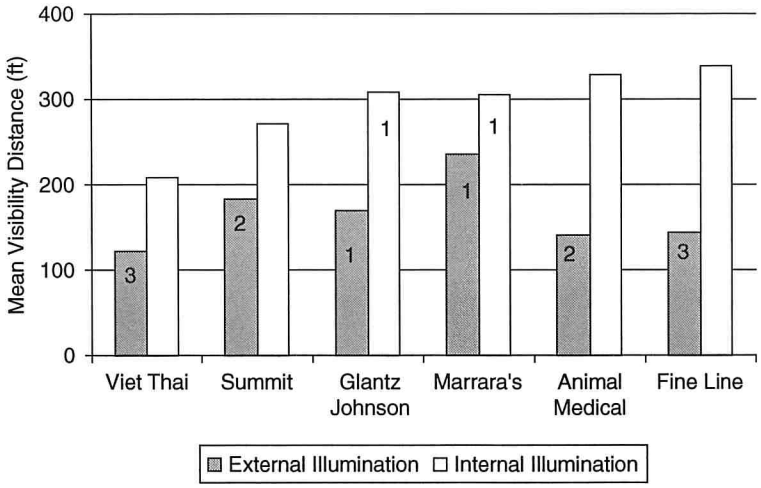


FIGURE 7 Effect of illumination type on individual sign visibility.

Results of this research clearly demonstrate the superiority of internally illuminated signs across a wide variety of driving conditions, sign offsets, sign sizes, shapes, colors, and external lighting designs and quality levels. Furthermore, internal sign illumination avoids some of the intractable problems with external illumination, illustrated in Figure 3, such as difficulty in maintaining the directionality of the light source over time, which often results in nonuniform light distribution (e.g., Viet Thai); "hot spots," especially on metallic inlay signs [e.g., Fine Line Homes, for which the luminance on the house icon reached more than 3,000 candela per square meter ($cd/m^2$)]; and light trespass, both onto other properties and into the eyes of oncoming drivers (e.g., Marrara's and Glantz, Johnson).

Although on-premise signs are a critical wayfinding device for drivers, poorly visible on-premise signs negatively affect road user safety by causing drivers to slow down in traffic (demonstrated in this research) or make erratic maneuvers. Internally illuminated on-premise signs have been shown to significantly increase the distance at which these signs can be read over externally illuminated signs. This was first demonstrated in a test track study in which 40% to 60% improvements were found. The present study showed that even greater improvements (almost 70% on average and 240% in the best case) can be made when in-use externally illuminated signs are upgraded to ones that use internal illumination. In this study, internally illuminated signs gave drivers on average about 2 s (and in extreme cases almost 4 s) more time than externally illuminated signs to read the signs and maneuver their vehicles.

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*The Visibility Committee peer-reviewed this paper.*

# Quantifying the Public Impacts of Highway–Rail Grade Crossings on Surface Mobility

## Regional Impact Model

Annie Protopapas, Stephen S. Roop, Curtis A. Morgan, Jeffery E. Warner, and Leslie E. Olson

**This research developed a model to evaluate the impacts associated with delayed vehicular traffic at highway–rail grade crossings in the eight-county area surrounding Houston, Texas. Vehicle traffic data, such as average daily traffic and distributions by roadway class and hour of day, were obtained from state and local agencies. Data from the operating railroads were used to determine train operating levels on an hourly basis for each rail corridor. Grade crossing data and unit costs were obtained from public sources. The developed model calculated the quantitative and monetary costs of mobility, fuel consumption, safety, and air quality impacts associated with vehicular impedance at rail grade crossings. It defined the existing (baseline) mobility constraints associated with train operations in the study region and provided a method to compare the impacts of potential infrastructure projects, changes in train operations, or both with the public benefits accruing from those changes during 10- and 20-year periods. Public benefits can reach significant levels and may take the form of a reduction in the cost burden associated with road traffic delays induced by rail operations. The model has been applied in a regional rail freight** study and has the potential for application in a host of other locations; this model will allow for the evaluation of investments in rail and highway **infrastructure or operations, ultimately ensuring that public benefits are commensurate with public costs.**

The ultimate objective of this model was to analyze and evaluate the effect of changes to railroad-related infrastructure and operations on surface mobility, safety, emissions, and fuel use. The model consisted of two key elements—an impedance component and an emissions component—that were linked with the latter dependent on the former. The impedance algorithm estimated primarily changes in vehicular operations—acceleration, deceleration, cruise, and idle durations and vehicle miles traveled (VMT)—as a result of at-grade rail crossing blockages. It subsequently estimated the associated delay, fuel consumption, and safety effects. The emissions model combined the changed vehicular operations with emission rates to determine the change in vehicular emissions as a result of the presence of trains at grade crossings. The model allowed for flexibility to estimate delay and emissions effects at various levels of spatial and temporal aggregation, as well as to perform numerous sensitivity analyses. Other uses included assistance in the estimation of factors such as noise. Figure 1 illustrates the relationship between the impedance and emissions components of the model.

The model included approximately 1,200 at-grade crossings in the Houston, Texas, region that resided on railroad main-line infrastructure, shown in Figure 2. The potential for significant interaction between vehicular and train operations across this heavily populated region is evident.

## IMPEDANCE MODEL

The impedance model used crossing-specific vehicle traffic data, for example, hourly traffic volumes, vehicle mix, speeds, capacities, and directional split, which were obtained from various sources including the Texas Department of Transportation, travel demand models, Houston's Major Thoroughfare and Freeway Plan, and previous Texas Transportation Institute (TTI) studies. They were used in conjunction with railroad operating data at each crossing, such as hourly train volume and train speed and length, as the determining factors to produce a measure for the number of vehicles, VMT, and time delayed at each intersection. To quantify vehicular delay, the model calculated the deceleration, idle, and acceleration components of vehicle operations when a train occupies a crossing and blocks traffic. Figure 3 provides a conceptual representation of the impedance model.

To develop the rail operational characteristics in the study area, rail activity data were obtained from the two major operating railroads in the vicinity—Union Pacific Railroad (UP) and BNSF Railway—which provided records of daily traffic data from a period in August 2005. UP provided a 17-day sample and BNSF provided a 31-day sample. The train movements contained in these records were condensed to an average typical day of operations over the railroad lines of interest in the study.

The railroads provided data from their respective computer-aided dispatching systems. These data came in the form of train movements past various "control points" (CPs), which read train identification data and record the time of day that a train passes. By accumulating

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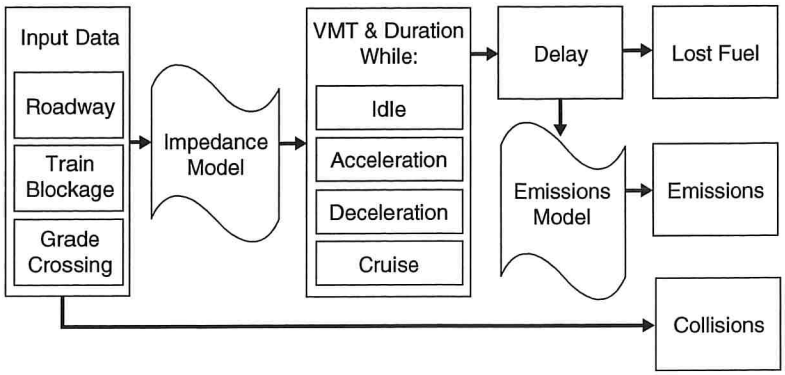


FIGURE 1 Impedance and emissions models concept.

this information, it was possible to determine the movement of trains past pairs of CPs and calculate the frequency, speed, direction of travel, and time of day that trains moved during the August sample period. Additional data were used to determine the length of each train operating at various points across the system. To assign crossing closure time, each highway–rail grade crossing was placed on the network relative to the CP pairs.

The model was designed to determine that portion of daily vehicle traffic affected by train traffic occurring during each 1-h period of the day. The analysis of the Houston region traffic patterns for road and rail showed that although vehicle traffic had discrete morning, noontime, and evening peaks, rail traffic was evenly distributed throughout the day. Therefore, there were periods during the early morning and late evening when there was relatively less conflict between vehicle and rail operations than found during peak periods of motorist activity. In simple percentage terms, only about 50% of train movements conflicted with most (80%) of the Houston region's vehicle traffic.

Queues are formed during train blockage, and these queues dissipate again as soon as the intersection is cleared. A 1-mi section was assumed for analysis, stretching half a mile in either direction from each crossing. This length was considered accurate and long enough to accommodate all possible queue lengths.

Additional data included grade crossing–specific data, such as the latest (2006) crash data (number of crashes, fatalities, and injuries by

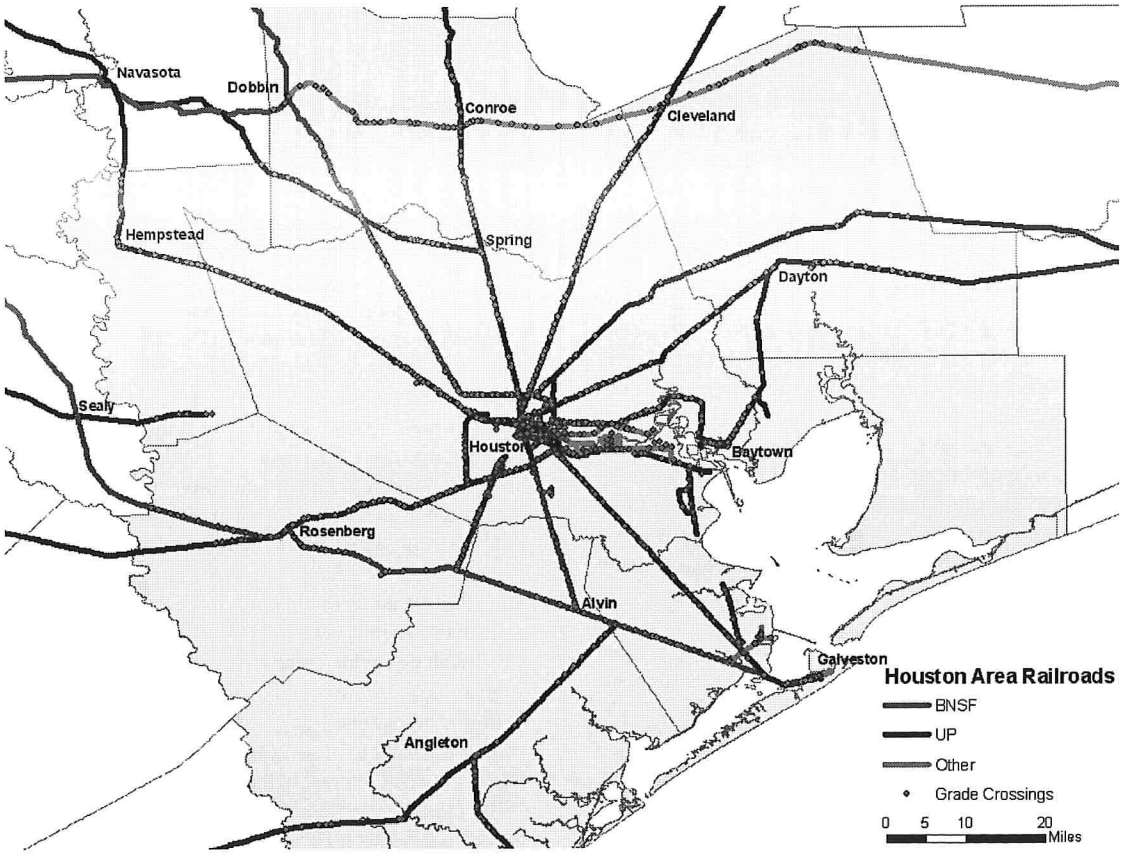


FIGURE 2 Grade crossing alignment across Houston region rail network.

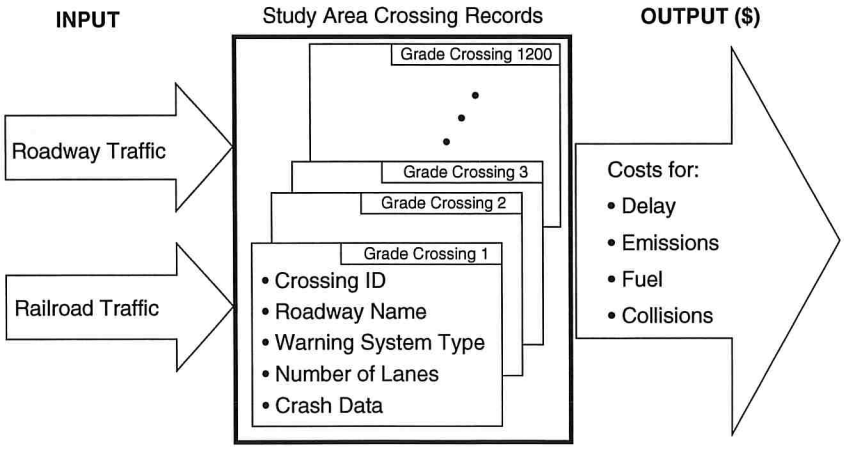


FIGURE 3 Impedance model concept.

severity) and warning system type, which were obtained from the Federal Railroad Administration's databases.

The initial calculation of costs associated with vehicular delay due to rail traffic incorporated current (most recent) year traffic volumes. These initial volumes were then projected by growth rates established for each corridor to arrive at a projected impact level during the analysis period. The Houston–Galveston Area Council (H-GAC) 2025 Regional Transportation Plan (RTP) provided 25-year vehicular growth estimates by employment center and corridor. Typically growth projections by corridor included increases in vehicle miles traveled within 2 to 5 mi on each side of major roadways. Annual traffic volumes at grade crossings were compounded accordingly because the alignment of these major roadways overlapped with the alignment of rail corridors in the study area. Increases in rail traffic volumes were assumed to be 3% annually. Figure 4 shows the expected pattern of traffic growth along several corridors based on the H-GAC 2025 RTP.

Thus the impedance model designated each grade crossing by rail corridor and by regional vehicular growth rate. This provided for traffic growth at each grade crossing during the project life. The economic effect of this increase in exposure at each grade crossing was measured in annual costs associated with vehicular delay, which consisted of vehicular emissions, lost time, wasted fuel, and safety. All unit values were the latest or national averages at the time the study was conducted:

- The emissions unit cost was obtained from the Texas Commission on Environmental Quality with oxides of nitrogen ($NO_x$) at $13,000/ton.
- Lost time unit cost was obtained from the consumer price index with $18.02/person-hour for passenger cars and light-duty trucks, and $76.29/person-hour for heavy-duty trucks.
- Fuel consumption unit cost was obtained from the American Automobile Association at $2.91/gal.
- Safety (collision) costs were obtained from the National Safety Council with fatalities at $3.76 million; injuries at $22,900, $48,200, and $188,000 according to severity; and property damage only, at $2,100.
- Total annual costs during 10- and 20-year periods were calculated in conjunction with a public discount factor (3%) to convert the costs incurred at each grade crossing in future years into an equivalent value in current-year dollars.

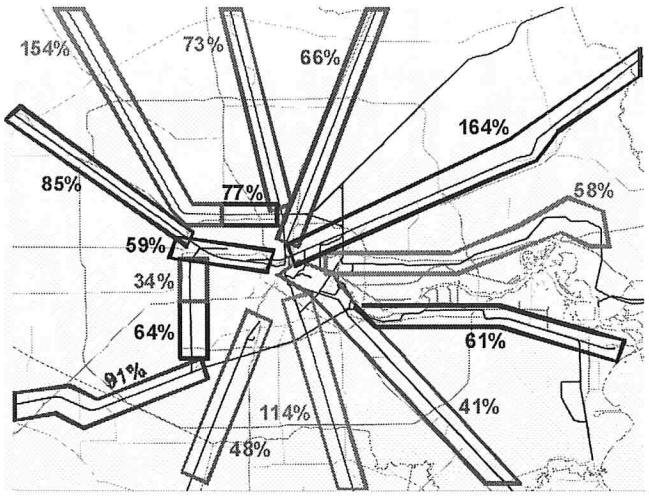


FIGURE 4 H-GAC 2025 VMT growth by corridor.

The model allowed for an assessment and evaluation of the degree to which public investment in rail-related infrastructure improvements could take place. Scenarios included infrastructure alterations such as grade separations, grade crossing closures, bypass rail routes, new rail connectors, double tracking, and extension of sidings. Rail operational changes included train speed or daily train traffic volume. Both types of improvements reduced the annual cost of vehicle–train interactions during the project life. These avoided costs represented the public benefit realized through public funding of projects that reduced delay at target grade crossings or otherwise altered the effect of train activity along corridors. It is possible that certain changes to operations or infrastructure may have reduced impedance at one location while increasing it at other(s). The model, with appropriate inputs, provided the output necessary to ascertain the net effects.

## EMISSIONS MODEL

The Houston–Galveston area is the Texas area with the most severe challenge in meeting the National Ambient Air Quality Standard for 8-h ozone established by the Environmental Protection Agency. Attainment in the Houston–Galveston area is especially challenging

because of the complex ozone formation chemistry and unique weather patterns in the area. The area is also highly populated with a total population of approximately 4.7 million and highly motorized with approximately 130 million daily VMT for an average ozone season weekday (Texas State Data Center). This eight-county area needs to meet ozone attainment by June 15, 2010 by sufficiently reducing the ozone precursors—$NO_x$ and volatile organic compounds (VOC). The emissions control strategies considered to date have not included possible emissions benefits due to strategies related to at-grade rail crossings such as grade separations, altered train operations, increased train speeds, train rerouting, or road rerouting. The emissions model was designed to help assess the air quality implications of various strategies related to at-grade rail crossings (*1*).

The following emissions-related data were used to build on the output of the impedance model to arrive at point-specific emissions impacts:

- Vehicle types and corresponding VMT mix values (percentage of VMT by vehicle type), which were aggregated into three categories—light-duty vehicles (LDVs), light-duty trucks (LDTs), and heavy-duty trucks (HDTs);
- Emission factors based on the latest (2002) attainment demonstration inventories performed by TTI for summer weekday and for each of the eight counties (units for all three pollutants—VOC, carbon monoxide, and $NO_x$—expressed in g/mi);
- Cruising emission rates, originally developed in 1.0-mph increments, allocated to the 5.0-mph MOBILE6 speed increments by interpolation and aggregated into four time periods—evening, morning peak, midday, and evening peak;
- Idling emission rate taken to be the MOBILE6 rate at the lowest speed (2.5 mph); and
- Emission rates for accelerations and decelerations calculated from portable emissions measurement system data collected by TTI and Clean Air Technologies.

The acceleration and deceleration rates for LDVs and LDTs were found to be approximately the same at 3.3 and 6.8 mph/s, respectively. The acceleration and deceleration rates for HDTs, however, were found to be 1.0 and 2.0 mph/s, respectively. The emission rates for all pollutants increased when vehicles accelerated and decreased when vehicles decelerated (as compared with cruising). LDVs and LDTs showed the greatest increase in relative emissions during acceleration, whereas the decrease during deceleration was fairly similar among the vehicle types. Given these relationships, emission rates were converted from grams per mile to grams per hour and from cruising emission rates to acceleration or deceleration emission rates. The resulting equation estimated emissions for each crossing by direction, time period, pollutant, and vehicle type, in grams, during train blockage. The same equation also estimated emissions with no blockage (only cruising), yielding a net emissions factor. Figure 5 shows the typical pattern of emissions for all vehicle types combined during each of the four operating regimes observed while a grade crossing was blocked as a result of a passing train.

## RESULTS

The model showed that more than 186,000 Houston-area motorists were delayed each day at one or more of the 1,200 grade crossings included in the study, resulting in almost 2 million unproductive hours per year and 584 tons of additional emissions per year.

During a 10-year period, given growth rates for both vehicle and train traffic, the total public cost of delay at these grade crossings in the eight-county study area was estimated to be $907 million. An examination by individual cost segment during a 10-year period estimated the cost of lost time at $758 million; the cost of collisions at more than $81 million; and the combined cost of emissions and wasted fuel at $68 million. Extending the analysis period to 20 years generated a total public cost of more than $2.6 billion, which included

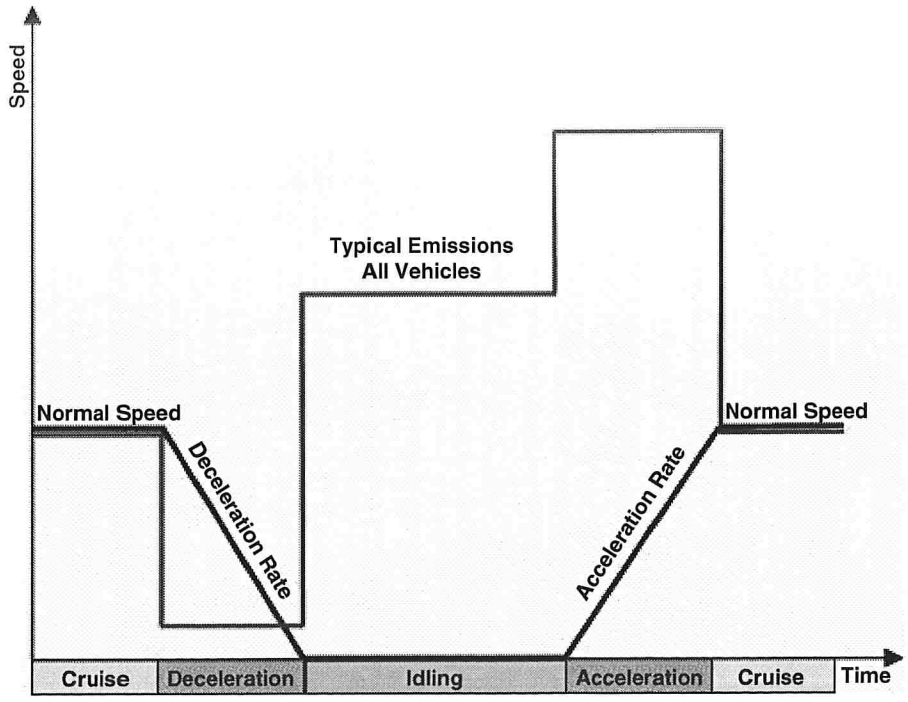


FIGURE 5 Typical emissions and speed regime versus time during train blockage.

$2.3 billion for lost time, $146 million for collisions, and $191 million for emissions and wasted fuel.

## CONCLUSIONS

There are significant levels of various types of public costs associated with vehicular impedance at highway–rail grade crossings, ranging from delay, to emissions, to wasted fuel, to collisions. The model uses local, specific data to produce more accurate results than models based on input values derived from national averages. It also provides the flexibility to test several scenarios and perform several sensitivity analyses during a project's life. It was used in a subsequent study, the Houston Region Freight Rail Study, to arrive at a list of short-range, medium-range, and long-range identified improvements (*2*). That helped the legislature understand the extent of the state's rail infrastructure needs and the associated investment required. Potential infrastructure projects could include grade separations, grade crossing closures, bypass rail routes, new rail connectors, double tracking, and extension of sidings; or there could be rail operational changes, such as train speed or daily train traffic volume. The model is spatially flexible and adaptable and can be applied in other locations, with the appropriate local input data. It can allow evaluation of any combination of rail and highway infrastructure investments, which can potentially reduce the effects associated with at-grade crossing blockages to ensure that public benefits are commensurate with public costs.

## ACKNOWLEDGMENTS

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# Speed Reduction Profiles Affecting Vehicle Interactions at Level Crossings with No Trains

Oi Kei Ng and Frank F. Saccomanno

**Observed road vehicle speed and deceleration profiles at active level crossings in the absence of a train are investigated and linked to safety performance. Results indicated that speed and deceleration profiles could be segmented into two zones along the approach road. Speed reduction was initiated at an upstream point along the road about 60 m from the track (Zone 1), and this became more pronounced for the road segment 20 to 30 m before the track (Zone 2). Although the deceleration rates based on average vehicle speeds were found to be within comfortable thresholds in both approach zones, a safety concern was raised for the worst-case scenario in which lead vehicles were reducing their speeds at a higher rate than following vehicles. Safety performance using an average crash potential index per vehicle was found to increase in the vicinity of the track for a given case study crossing application. The range of safety performance values was also found to be greater in Zone 2; this finding suggested possible safety concerns about rear-end crashes taking place near the track and the added possibility of a vehicle being pushed onto the track in the path of an oncoming train. This study provides useful insight for modifying speed profiles in a traffic simulation model to account for the presence of a level crossing. The analysis also provides insights into a base crash risk for an open crossing. These insights can be used to refine previous estimates of countermeasure effectiveness, including closures.**

The potential for rear-end crashes results from disruptions in traffic flow or turbulence along a given road segment. Grade crossings, like intersections, are geometric features that "obstruct" traffic flow and increase the level of turbulence. This assertion is supported with reference to historical crash experience at grade crossings, at which 75% of reported crashes resulted from rear-end impacts between vehicles, notwithstanding the absence of a train (*1, 2*). When a train is present, the severity of these types of crashes can be especially high if they lead to vehicles becoming disabled on the track in the path of the oncoming train.

Much of the research on grade crossing safety has centered on potential crashes between road vehicles and oncoming trains (*3, 4*). The mechanism whereby grade crossings produce increased turbulence in the traffic stream with increased potential for rear-end crashes is not well understood. Traffic turbulence at grade crossings differs slightly from turbulence at intersections. Both take place as a result of specific vehicles reducing their speed at some critical distance initiating a chain reaction of braking involving subsequent vehicles in the traffic stream. At grade crossings, however, there are two reasons that drivers may reduce their speeds despite the absence of a train: (*a*) confirmation that despite inactive signals at the crossing a train is indeed absent and (*b*) track "speed bump" effect. Recent studies suggest that despite the absence of trains, drivers approaching a crossing will reduce their speeds in the vicinity of the track, with increased vehicle interaction and traffic flow turbulence.

This paper has three basic objectives:

1. Describe average vehicle speed and deceleration profiles at open status crossings with respect to distance from the track as reported in previous studies;
2. Obtain real-time vehicle speeds and deceleration rates from videotaped crossing data, and compare these results with values reported in the literature and with desirable thresholds for safe traffic operations; and
3. Use an existing real-time safety performance measure to investigate the potential for paired vehicle interactions as a function of distance to the track.

The results of this analysis should provide some meaningful insights as to how safety can be compromised in the vicinity of a grade crossing without the presence of a train, based on the potential for rear-end vehicle interactions. Furthermore, the paper identifies the existence of distance zones near level crossings that require adjustment in vehicle speed profiles for traffic simulation notwithstanding the presence of a train. Existing traffic simulation packages do not consider the speed issue for level crossings. The results of this paper provide a reliable method for estimating countermeasures effectiveness especially as they relate to closures.

## SPEED PROFILES AS REPORTED FROM THE LITERATURE

The profile of vehicle speeds with respect to distance to the crossing is not well understood. The current literature has suggested that drivers tend to reduce their speeds in the vicinity of a crossing to avoid the impact of an uneven pavement at the track or because of uncertainty about the presence of a train or for both reasons (*1, 5, 6*). Several important factors need to be determined: the magnitude of the speed reduction, distance from the track at which this reduction is initiated, the effect of distance on speed reduction, and average deceleration rates.

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Four recent experimental studies have reported significant speed reductions for vehicles approaching a crossing at different distances from the track. This reduction takes place despite the absence of a train at the crossing. Tenkink and Van der Horst studied the speed profiles of approaching road vehicles at two rural crossings equipped with flashing lights (red with train, white without train) in the Netherlands (*1*). Coleman and Moon investigated speed profiles at two grade crossings, both situated along the Chicago, Illinois–St. Louis, Missouri, high-speed rail corridor in the United States (*5*). Ward and Wilde investigated speed profiles for vehicles approaching a rural two-lane passive crossing (crossbuck only) in Central Ontario (*6*).

In the Tenkink and Van der Horst study, under a white signal drivers were permitted to traverse the tracks without slowing down, whereas under a red signal drivers were required to come to a full stop (*1*). The two-phase signal (red and white) in this study added an additional level of control over conventional single-phase signals (red only), in that drivers were formally informed that no train was approaching the crossing. In this study, the site was videotaped from 8 a.m. to 6 p.m. during a 7-day period. Speed and deceleration profiles were estimated in approximately 5-m increments beginning at a distance of 70 m from the track.

In the Coleman and Moon study, speed profiles were obtained from vehicle samples videotaped at two dual gate–equipped sites (*5*). The Hartford, Illinois, site is an industrial crossing with a posted speed limit of 45 mph (72 km/h). The McLean, Illinois, site along US-136 has a posted speed limit of 40 mph (64 km/h). In these studies, pavement markings were used as location reference points, and the videotape profiles were coded into three zones with respect to distance from the track—Zone A: distance 93 to 31 m (at the McLean site) and 77 to 31 m (at the Hartford site), Zone B: distance 31 m to entry barrier, and Zone C: distance 28 m between entry to exit barriers on either side of the track.

In the Ward and Wilde study, two passive crossings were investigated with a posted road speed limit of 60 km/h (*6*). Sonar units were stationed along the roadside at incremental distances varying in length from 5 m to 25 m, and speeds were measured from soundings. The resultant average speeds were reported for seven zones with respect to distance from the track. The mean speed profiles obtained from the four studies above are illustrated in Figure 1, along with their reported corresponding ±1 standard deviation (SD) confidence interval. For the Ward and Wilde study, only mean values were reported (*6*). The ±1 SD confidence interval provides an indication of the range of speeds observed at the various reference points (distance) for the entire sample of vehicles in each study. In general, vehicle-specific speeds encompass a range of values between ±6 m/s over the entire approach segment, and this range does not appear to be affected by distance to the track.

As indicated in Figure 1, active crossings are defined as being equipped with either flashing lights and bells or flashing lights, bells,

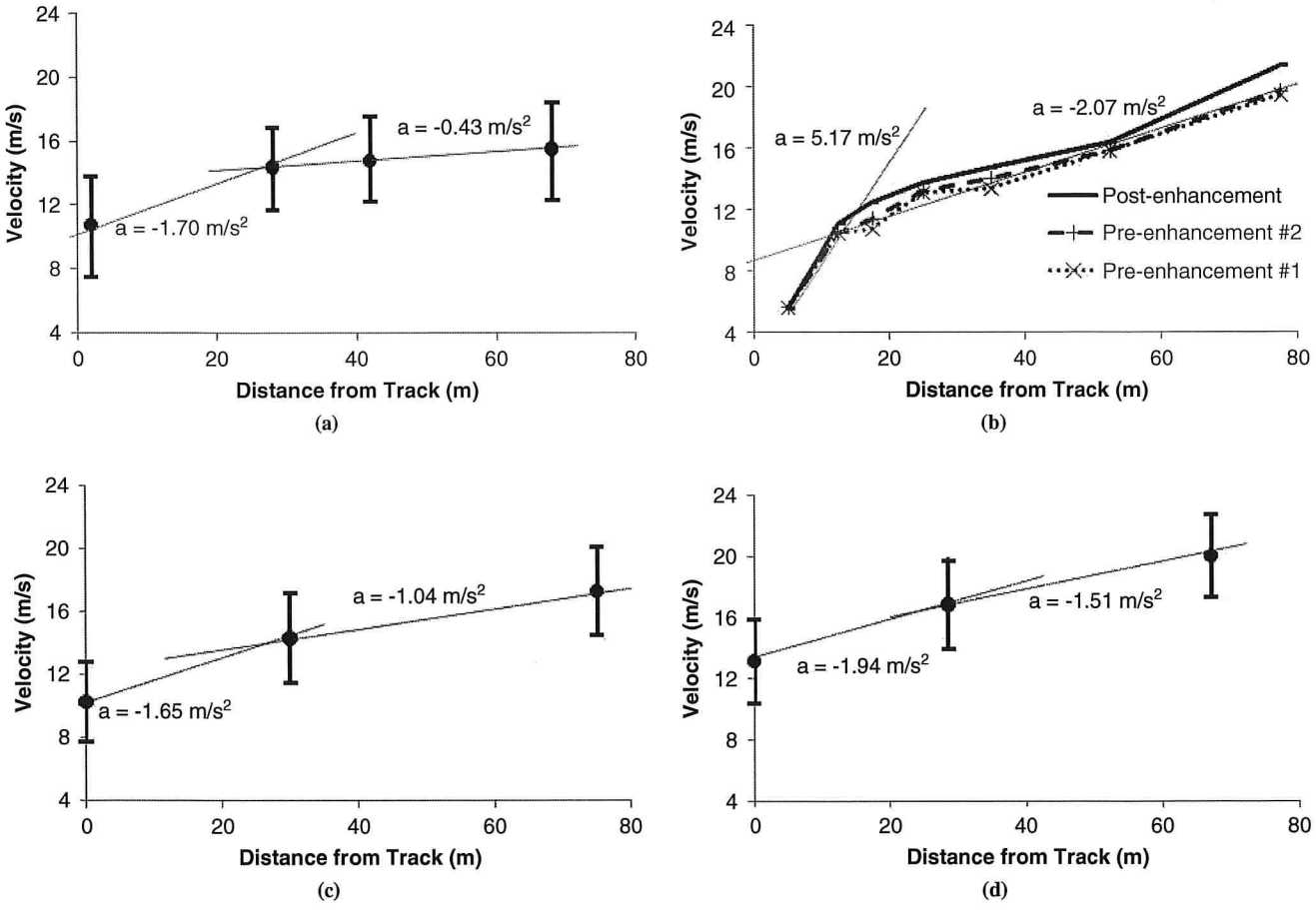


FIGURE 1 Speed profiles from three active crossings and one passive crossing: (*a*) AKI-I, Netherlands (50 km/h); (*b*) test site, Central Ontario (60 km/h) (passive); (*c*) McLean, U.S.–Group C, single (64 km/h); and (*d*) Hartford, U.S.–Group A, single (72 km/h).

and gates. Passive crossings are defined as being equipped with standard reflective crossbuck or other fixed signs.

Studies from the active crossings [excluding Ward and Wilde (*6*)] are consistent in suggesting that in the absence of trains, road vehicles on average will reduce their speeds with distance to the track. All studies report a similar track crossing speed of between 10 and 13 m/s (approximately 35 and 50 km/h). These studies seem to be inconsistent as to the maximum distance from the track at which speed reduction is initiated. Coleman and Moon (*5*) suggested a maximum distance of about 70 m [similar to Ward and Wilde (*6*)], whereas Tenkink and Van der Horst suggested that the initial speed reduction became significant only at a distance of about 30 m from the track (*1*). Before the 30-m reference point, vehicle deceleration rates were found to be low such that vehicles were considered as maintaining a uniform approach speed. The resultant average speed profiles obtained from these studies are illustrated in Figure 1 with means, ±1 SD, and corresponding deceleration rates at various distances from the track.

## Magnitude of Speed Reduction

There is a significant difference between active and passive grade crossings in regard to the magnitude of speed reduction. The average speed recorded at the track was found in the studies above to be about 35 km/h for all active grade crossings. For the passive crossing considered by Ward and Wilde with a posted speed limit of 60 km/h, a 70% speed reduction was observed from 19 m/s (68 km/h) to 5 m/s (20 km/h) (*6*). For the active crossings with posted speed limits of 50 km/h, 64 km/h, and 72 km/h, the percentage reduction in speed was found to be 30.6%, 48.1%, and 40.7%, respectively.

Because the magnitude of speed reduction is greater for roads with higher posted speed limits, we would expect a higher deceleration rate at these crossings with a corresponding higher number of rear-end vehicle interactions. In this paper, this will be investigated further with regard to distance-related average deceleration rates.

## Zonal Segmentation of Speed Reduction

In general, studies documented in the literature found that there is a gradual speed reduction in the upstream segment (Zone 1: more distant segment from the track) followed by a more abrupt higher speed reduction in the downstream segment (Zone 2: nearer to the track).

For the passive crossing investigated by Ward and Wilde, an average deceleration rate of 2.1 m/s (7 km/h) estimated for Zone 1 is followed by a considerably higher deceleration rate of 5.2 m/s (18 km/h) in Zone 2 (*6*). Differences in the average deceleration rates were found to be especially abrupt at a distance of about 10 to 15 m from the track. The majority of the speed reductions observed by Coleman and Moon were also found to take place within the first 15 m of the track, with an average rate of 1.8 m/s (7.2 km/h) (*5*). If we assume that the initial reduction in speed was initiated at about 60 m from the track, the deceleration rate in Zone 1 would vary between 1.0 m/s (3.6 km/h) and 1.5 m/s (5.4 km/h).

For a posted speed of 50 km/h, Tenkink and Van der Horst found that most of the speed reduction took place within the first 30 m from the track, with a rate of 1.7 m/s (6.1 km/h) (*1*). This study noted that a more gradual reduction in speed took place in the upstream segment (Zone 1) with a rate of about 0.4 m/s (1.5 km/h). A more gradual reduction in speed was observed at Zone 1, which is consistent with the results reported by Coleman and Moon for the U.S. crossings (*5*).

## Analysis of Vehicle Deceleration Profiles

Changes in deceleration rates are an important aspect of safety performance. For each speed profile, deceleration rates are estimated on the basis of differences in average speed between vehicles, as obtained at adjacent reference points. The worst-case estimate was obtained by comparing the +1 SD upperbound value at the upstream reference point for the following vehicle to the −1 SD lowerbound value at the downstream reference point for the lead vehicle. These values are summarized in Table 1 at the three reported crossing sites, the Netherlands site from Tenkink and Van der Horst (*1*), and the Hartford and McLean sites from Coleman and Moon (*5*). As was illustrated previously in Figure 1, deceleration rates were estimated in 10-m segments from differential speed profiles at six distance reference points. Because Ward and Wilde did not provide upper- and lowerbound speeds, their study was not considered in our worst-case estimates of deceleration rates (*6*). Estimates of the worst-case deceleration rate are given in Table 1.

As expected, the worst-case deceleration rates reported at the three crossing sites were found to be considerably higher than rates based on mean speed differentials. The 10-m distance intervals do not seem to provide a sufficient level of discrimination to explain differences in vehicle deceleration rates with respect to distance to the track. However, when the 10-m intervals were aggregated into two zonal segments, as illustrated in Figure 1, the mean and worst-case

**TABLE 1 Mean and Worst-Case Deceleration Profiles from Three Active Crossings**

| | AKI-1, Netherlands | | Hartford, United States—Group A | | McLean, United States—Group C | |
|---|---|---|---|---|---|---|
| Distance from Track (m) | Deceleration (Figure 1) [m/s$^2$] | Deceleration (worst case) [m/s$^2$] | Deceleration (Figure 1) [m/s$^2$] | Deceleration (worst case) [m/s$^2$] | Deceleration (Figure 1) [m/s$^2$] | Deceleration (worst case) [m/s$^2$] |
| 60 | −0.43 | −8.82 | −1.51 | −12.41 | −1.04 | −10.08 |
| 50 | −0.43 | −8.40 | −1.51 | −11.80 | −1.04 | −9.67 |
| 40 | −0.43 | −8.18 | −1.51 | −11.42 | −1.04 | −9.26 |
| 30 | −1.70 | −9.04 | −1.94 | −11.81 | −1.65 | −9.45 |
| 20 | −1.70 | −8.93 | −1.94 | −10.69 | −1.65 | −8.28 |
| 10 | −1.70 | −8.36 | −1.94 | −9.56 | −1.65 | −7.14 |

TABLE 2 ANOVA Table

| | Average Speed | | | Deceleration at Worst Case | | |
|---|---|---|---|---|---|---|
| | Test 1 | Test 2 | Test 3 | Test 1 | Test 2 | Test 3 |
| Block level significant ($p$-value) | Yes (<.0001) | Yes (.0029) | Yes (<.0001) | Yes (<.0001) | Yes (.0176) | Yes (<.0001) |
| Treatment significant ($p$-value) | Yes (.0098) | Yes (.0052) | Yes (<.0001) | Yes (.0234) | Yes (.0375) | Yes (.0023) |

deceleration rates were found to vary considerably, such that higher rates were experienced in the segment near to the track. For the worst case, these deceleration rates were found to exceed the accepted "comfortable" threshold of 2.4 m/s$^2$ commonly suggested in the literature, by a factor of 5 (*7*).

Several ANOVA tests were carried out to investigate the statistical significance of distance to the track on average speed and worst-case deceleration rates. The test structures for average speeds and worst-case deceleration rates are given as

| *Test* | *Treatment Effect (distance from track)* |
|---|---|
| 1 (in Zone 1) | 60 m, 50 m, 40 m, 30 m |
| 2 (in Zone 2) | 20 m, 10 m, 0 m |
| 3 (between zones) | Zone 1 and Zone 2 |

In these tests, the corresponding block effects were established as the three crossing sites (Netherlands and the two U.S. sites, Hartford and McLean). The tests were carried out separately for average speeds and worst-case deceleration rates, with the result given in Table 2.

These ANOVA tests yielded a number of interesting observations:

- The block effect (crossing site) was found to explain a statistically significant amount of the variation in average speeds and worst-case deceleration rates.
- More important, variations in average speeds and worst-case deceleration rates were found to depend on distance from the track. With shorter distances, lower speeds were observed as well as higher worst-case deceleration rates. These relationships were found to be significant at the 5% level.
- For Test 3, statistically significant differences in both average speeds and worst-case deceleration rates were obtained between Zones 1 and 2 for the combined active crossing sample.

Observations based on average speed profiles (Figure 1 and Table 2) cannot address individual vehicle pair interaction, because this type of analysis requires observed vehicle specific speed and spacing profiles. The discussion has been based on traffic stream averages at different reference points with respect to the track. To obtain observed vehicle specific speed and spacing profiles it was necessary to collect vehicle tracking data from observational field experiments for a given crossing.

## SPEED PROFILES AT THE KING STREET CROSSING

The site selected for speed analysis consists of a single gate–equipped crossing intersecting a four-lane approach road (King Street) in Kitchener, Ontario, Canada, (as illustrated in Figure 2). The background photo was taken in 2006; pavement markings have been slightly modified to correspond to their current location for the data

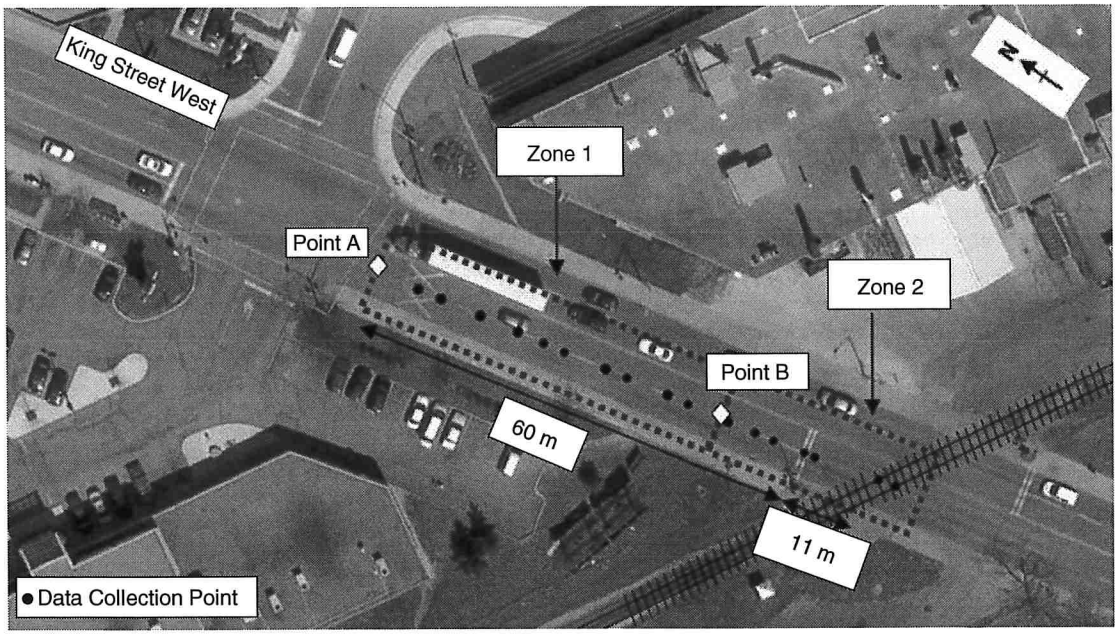


FIGURE 2 Site location for King Street crossing (background extracted from City of Kitchener interactive online Internet mapping).

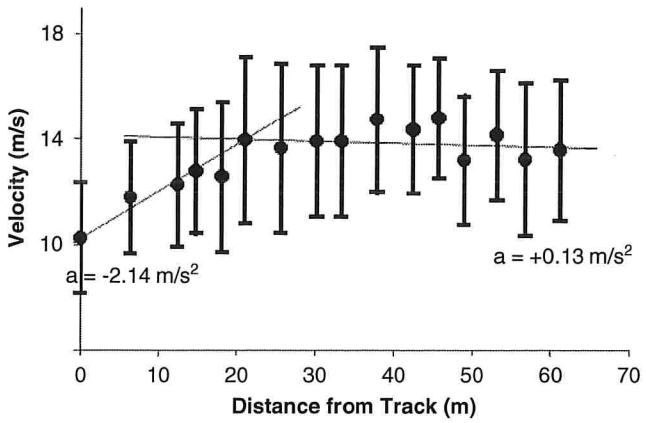


FIGURE 3 Speed profile from Kitchener site.

collection exercise. The King Street crossing was selected because of its relative isolation from other nearby intersections and driveways. The posted speed limit on King Street at this site is 50 km/h. Although a significant number of passenger and freight trains use this crossing, the data collected in this study were obtained when no train was present at the crossing.

The data collection consisted of a 15-min videotaping of traffic in the southbound direction along King Street. The speed of vehicles in the northbound direction was assumed to be affected by discharging vehicles from a major downstream signalized intersection. In this data collection, vehicles entering the approach segment from side streets and driveways were ignored, as were adjacent vehicles in the sequence. Only "uninterrupted" vehicles traversing Point A in Figure 2 were sampled. Because buses are required to come to a full stop at the crossing, they were also removed from the vehicle sample. The speed and progression of 53 vehicles (autos) were obtained in this exercise.

The southbound approach road was divided into two zones with regard to distance from the track: Zone 1 between Point A and Point B (about 20 m from the track) and Zone 2 between Point B and the track itself. Fixed objects such as lane dividers and tracks were used as reference points for estimating the distances. During video playback, time was recorded per vehicle on a frame-by-frame basis as each vehicle crossed a given reference point. On the basis of the time progression of vehicles in Zones 1 and 2, speed profiles were obtained at all reference points.

The speed profiles obtained from the King Street crossing are illustrated in Figure 3, with corresponding means and ±1 SD confidence intervals. There are 11 reference points along the approach road in Zone 1 (with a length of 40 m) and five reference points in Zone 2 (with a length of 20 m).

An average incoming speed of 50 km/h was observed at Point A, which can be compared with the average track crossing speed of 36 km/h. Vehicles were observed to maintain a uniform speed in Zone 1 with an average acceleration rate of +0.1 m/s$^2$ and to decelerate significantly in Zone 2 with an average rate of –2.1 m/s$^2$. In fact, for Zone 1 the mean deceleration rate was so low as to reflect a nearly uniform speed profile along this more distant segment.

Most of the speed reduction at King Street took place in the zonal segment nearer the track. The distance at which drivers initiated speed reductions could not be ascertained accurately with respect to measurements taken from the pavement markings. Drivers approaching the King Street crossing in the southbound direction did not reduce their speeds appreciably until they reached a point about 20 m from the track. This result is consistent with observations from Tenkink and Van der Horst (*1*), but inconsistent with Coleman and Moon (*5*) and Ward and Wilde (*6*). The latter studies indicated a gradual reduction in speed in Zone 1. The worst-case deceleration rates with distance from the track were estimated to be –7.2 m/s$^2$ (60 m), –6.9 m/s$^2$ (50 m), –9.1 m/s$^2$ (40 m), –8.8 m/s$^2$ (30 m), –8.7 m/s$^2$ (20 m), –6.9 m/s$^2$ (10 m).

An ANOVA test using the zone and distance categories discussed above was carried out to investigate the statistical significance of average speed and worst-case deceleration rates for the King Street crossing; results are summarized in Table 3. In this analysis only one site has been considered, hence there are no "block effects" in the ANOVA. Because of insufficient data for Zone 2, within-zone comparisons by distance interval could not be carried out.

The King Street ANOVA yielded several important observations:

- For Test 1, differences in average speeds and worst-case deceleration rates in Zone 1 were not found to be statistically significant at the 5% level. These results are inconsistent with speed reduction profiles in Zone 1 as reported in the U.S. studies.
- For Test 3, the differences in the worst-case deceleration rate between Zones 1 and 2 were not found to be significant. The reason could be that the ±1 SD difference in speed between lead and following vehicle pairs does not vary between the zones. The differences in speed reduction in the grouping of aggressive and nonaggressive drivers in this sample were not significant.
- In Zone 2, the higher worst-case deceleration rate could have serious safety implications that need to be addressed.

TABLE 3 ANOVA Result from King Street Crossing

| | Average Speed | | | Deceleration at Worst Case | | |
|---|---|---|---|---|---|---|
| | Test 1 | Test 2 | Test 3 | Test 1 | Test 2 | Test 3 |
| Treatment levels | 60 m<br>50 m<br>40 m<br>30 m<br>20 m | 10 m<br>0 m | Zone 1<br>Zone 2 | 60 m<br>50 m<br>40 m<br>30 m<br>20 m | 10 m | Zone 1<br>Zone 2 |
| Treatment significant ($p$-value) | No (.7941) | N/A[a] | Yes (.0060) | No (.1410) | N/A[a] | No (.3057) |

[a]Insufficient data point.

## MEASURING SAFETY PERFORMANCE

In this section, time-dependent vehicle speed, spacing, and deceleration profiles are used as inputs into measures of safety performance. A number of safety performance measures have been introduced in the safety literature, including time to collision, deceleration rate to avoid the crash (DRAC), and postencroachment time (*8*). Archer has explicitly recognized the relevance of DRAC as a safety performance measure (*9*). This measure considers the role of speed differentials and decelerations in crash occurrence and is expressed as a function of the time–space and deceleration profiles experienced by individual vehicle pairs in the traffic stream (lead and following vehicles).

Researchers have argued that the conventional DRAC measure fails to accurately reflect traffic conflicts and, hence, identify potential crash situations because it does not consider vehicle braking capability over time for prevailing road and traffic conditions. For example, it is expected that DRAC under wet pavement conditions is more critical to safety than the same DRAC under dry pavement conditions (with higher pavement–tire friction), but the actual measure does not take pavement surface condition and braking into account.

In their investigation of crash risk at intersections, Cunto and Saccomanno defined a crash potential index (CPI) in regard to the probability that a given vehicle DRAC exceeds its maximum available deceleration rate (MADR) or braking capability for every 0.1 s of simulated time (*10*). The CPI was obtained from an expression of the form

$$\mathrm{CPI}_i = \frac{\sum_{t=\mathrm{ti}_i}^{\mathrm{tf}_i} P\left(\mathrm{DRAC}_{i,t} > \mathrm{MADR}_{i,t}\right) \cdot \Delta t \cdot b}{T_i} \quad (1)$$

where

$\mathrm{CPI}_i$ = crash potential index for vehicle $i$,
$\mathrm{ti}_i$ = initial time interval for vehicle $i$,
$\mathrm{tf}_i$ = final time interval for vehicle $i$,
$\Delta t$ = observation time interval (s), and
$T_i$ = total simulated time for vehicle $i$ (s).

The term $b$ in the expression above denotes a binary state variable, 1 if $\mathrm{DRAC}_{i,t} > 0$ and 0 otherwise. This suggests that a conflict can occur only when the following vehicle is approaching the lead vehicle. In this analysis, MADR is assumed to be a truncated normal distribution with an average of 8.45 m/s$^2$ and standard deviation of 1.40 m/s$^2$ (*10*).

A safety performance profile was obtained for the King Street crossing based on the average and 90th percentile values of CPI per vehicle as extracted from the video data for the two crossing zonal segments. The measure of CPI required a pairing of following to lead vehicles from the individual vehicle profiles. The results are illustrated in Figure 4.

Higher levels of CPI per vehicle reflect lower safety performance. A number of observations were obtained from this figure:

- Both average and 90th percentile measures of CPI per vehicle are significantly higher in Zone 2 nearer to the track, and this result is due in large part to differences in vehicle speed and deceleration rates between the two zonal segments. The 90th percentile value in Zone 1 is closer to the mean value obtained for Zone 2.

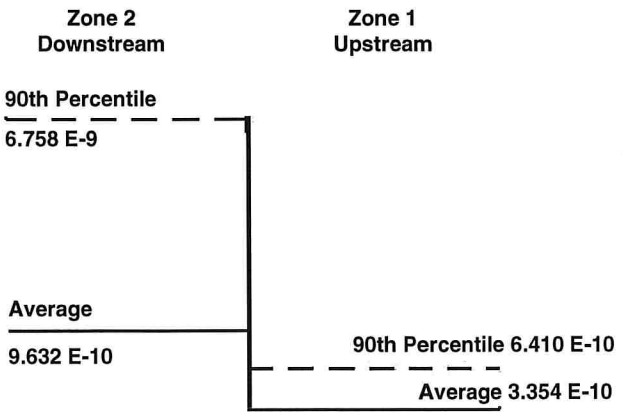


FIGURE 4 Comparison of CPI per vehicle between upstream and crossing area.

- Variation of the mean CPI per vehicle in Zone 2 is considerably higher than in Zone 1, suggesting that there is a wider range of vehicle interactions in this zone that could compromise safety. For Zone 2 a few vehicle pairs were observed to experience unusually high levels of risk (6.758 E-9). The narrower range of CPI per vehicle values in Zone 1 suggests a less abrupt speed reduction response from individual vehicles and a reduced chance of unsafe rear-end interactions.
- In Zone 2, the level of crash risk increases with distance to the track, whereas in Zone 1 the values of CPI per vehicle were found to be fairly uniform.
- As in previous studies from the literature, the King Street speed profile fails to provide a definitive point upstream of the crossing at which speed reduction is initiated. An increase in CPI per vehicle is experienced primarily in the 30-m segment nearest to the track (Zone 2).
- Because crash risk is expected to be higher in Zone 2 (higher CPI per vehicle), the possibility of vehicle entrapment between barriers at gate-equipped crossings, vehicles being "pushed" onto the track, or vehicles becoming disabled on the track becomes especially problematic. Any evaluation of crossing countermeasures should consider those risks.

## CONCLUSIONS

Consistent with driving behavior as inferred from previous studies, the speed profile from the King Street crossing demonstrates a two-zone transition. The change of speed in all active crossings is mild at the upstream segment starting at about 60 m from the crossing and becomes more abrupt at the 20- to 30-m segment nearer to the track. Although on average deceleration rates are below the comfortable thresholds, a safety concern is raised for individual vehicle pairs (following–lead vehicles) when the worst-case speed reduction profiles are considered. In this case, deceleration rates in Zone 2 can be as much as five times higher than the comfortable threshold. This poses some significant safety concerns at level crossings, despite the absence of a train.

Observational results obtained from the King Street crossing are consistent with average speed reduction values reported in the literature. The CPI-based safety performance measure estimated in

this study suggests an increased chance of rear-end vehicle interactions in proximity to the track. This risk occurs despite the absence of a train, and this provides important insights for investigation of vehicle entrapment and signal preemption.

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*The Highway–Rail Grade Crossings Committee peer-reviewed this paper.*

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JOURNAL OF THE TRANSPORTATION RESEARCH BOARD

# Peer Review Process

The ***Transportation Research Record: Journal of the Transportation Research Board*** publishes approximately 25% of the more than 3,000 papers that are peer reviewed each year. The mission of the Transportation Research Board (TRB) is to disseminate research results to the transportation community. The Record series contains applied and theoretical research results as well as papers on research implementation.

The TRB peer review process for the publication of papers allows a minimum of 30 days for initial review and 60 days for rereview, to ensure that only the highest-quality papers are published. At least three reviews must support a committee's recommendation for publication. The process also allows for scholarly discussion of any paper scheduled for publication, along with an author-prepared closure.

The basic elements of the rigorous peer review of papers submitted to TRB for publication are described below.

## Paper Submittal: June 1–August 1

Papers may be submitted to TRB at any time. However, most authors use the TRB web-based electronic submission process available between June 1 and August 1, for publication in the following year's Record series.

## Initial Review: August 15–November 15

TRB staff assigns each paper by technical content to a committee that administers the peer review. The committee chair assigns at least three knowledgeable reviewers to each paper. The initial review is completed by mid-September.

By October 1, committee chairs make a preliminary recommendation, placing each paper in one of the following categories:

1. Publish as submitted or with minor revisions,
2. Publish pending author changes and rereview, or
3. Reject for publication.

By late October, TRB communicates the results of the initial review to the corresponding author indicated on the paper submission form. Corresponding authors communicate the information to coauthors. Authors of papers in Category 2 (above) must submit a revised version addressing all reviewer comments and must include a cover letter explaining how the concerns have been addressed.

## Rereview: November 20–January 25

The committee chair reviews revised papers in Category 1 (above) to ensure that the changes are made and sends the Category 2 revised papers to the initial reviewers for rereview. After rereview, the chairs make the final recommendation on papers in Categories 1 and 2. If the paper has been revised to the committee's satisfaction, the chair will recommend publication. The chair communicates the results of the rereview to the authors.

## Discussions and Closures: February 1–May 15

Discussions may be submitted for papers that will be published. TRB policy is to publish the paper, the discussion, and the author's closure in the same Record.

Many papers considered for publication in the *Transportation Research Record* are also considered for presentation at TRB meetings. Individuals interested in submitting a discussion of any paper presented at a TRB meeting must notify TRB no later than February 1. If the paper has been recommended for publication in the *Transportation Research Record*, the discussion must be submitted to TRB no later than April 15. A copy of this communication is sent to the author and the committee chair.

The committee chair reviews the discussion for appropriateness and asks the author to prepare a closure to be submitted to TRB by May 15. The committee chair reviews the closure for appropriateness. After the committee chair approves both discussion and closure, the paper, the discussion, and the closure are included for publication together in the same Record.

## Final Manuscript Submittal: March 15

In early February, TRB requests a final manuscript for publication—to be submitted by March 15—or informs the author that the paper has not been accepted for publication. All accepted papers are published by December 31.

## Paper Awards: April to January

The TRB Executive Committee has authorized annual awards sponsored by Groups in the Technical Activities Division for outstanding published papers:

- Charley V. Wootan Award (Policy and Organization Group);
- Pyke Johnson Award (Planning and Environment Group);
- K. B. Woods Award (Design and Construction Group);
- Patricia F. Waller Award (Safety and System Users Group);
- D. Grant Mickle Award (Operations and Preservation Group); and
- John C. Vance Award (Legal Resources Group).

Other Groups also may nominate published papers for any of the awards above. In addition, each Group may present a Fred Burggraf Award to authors 35 years of age or younger.

Peer reviewers are asked to identify papers worthy of award consideration. Each Group reviews all papers nominated for awards and makes a recommendation to TRB by September 1. TRB notifies winners of the awards, which are presented at the following TRB Annual Meeting.

**Transportation Research Board**
**www.TRB.org**

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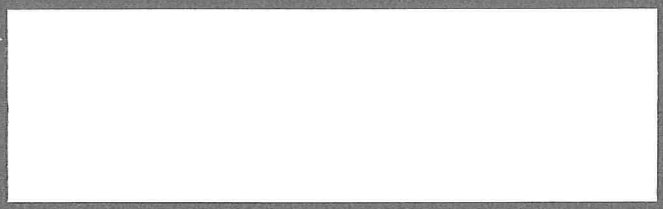

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